EVALUATION AND DESIGN FOR GRADE CONTROL
TASK COMMITTEE

SITING, MONITORING, & MAINTENANCE FOR THE DESIGN OF GRADE CONTROL STRUCTURES

FINAL REPORT

<FRONT COVER PICTURE PLACEHOLDER>

PREFACE

Urbanization, mining, agriculture, construction, and other human activities have led to the destabilization of many streams and rivers. The destabilized condition of these rivers and streams contributes to erosion and sedimentation and the loss of valuable land resources. One of the key influences in reestablishing stream stability is providing stream bed elevation or grade control.

The Hydraulic Structures Committee authorized the Grade Control Task Committee in 1995. They charged the Grade Control Task Committee to prepare a comprehensive review of related literature, organize technical sessions on grade control at appropriate ASCE conferences, and to compile a state of the art report detailing criteria for siting, structure selection, and design of grade control.

The committee acknowledges the contributions and assistance of …

<Please note that the brackets <> indicate my editorial markers throughout this draft.>

PURPOSE

Since 1970, numerous grade control and energy dissipation structures have been developed to stabilize stream beds and banks. Currently, there is not a single document or reference that consolidates the multitude of fragmented information that presents selection and/or design criteria of the available features. In addition, little guidance has been developed that includes structure siting into the selection and design process. It is anticipated that as environmental, habitat and aesthetic features are integrated into the stream restoration/rehabilitation processes, it will be important for the new generation of design water resource engineers to have a comprehensive reference for grade control alternatives.

<Needs more but it is a start>

TASK COMMITTEE OBJECTIVES

The objectives of the Task Committee are:

(1) Conduct a comprehensive review of existing grade control feature Literature;

(2) Organize one or more conference sessions for the 1996 Water Congress.

(3) Compile a state-of-the-art report to include: a) how to determine the peed for grade control, b) siting of grade control features, c) alternative grade control features available to the design engineer, d) criteria for selecting the appropriate grade control feature, e) general grade control feature design criteria; and f) references for specific grade control feature designs.

COMMITTEE MEMBERSHIP

Phil Combs

Chester Watson

Chester Watson is an Assistant Professor of Civil Engineering at Colorado State University. He specializes in the analysis of watershed and channel system instability. Dr. Watson is an expert in the causes of watershed and river instability, reduction of sediment production, and channel rehabilitation. He leads seminars at the Natural Resources Conservation Service (formerly the Soil Conservation Service) and US Army Corps of Engineers on fluvial geomorphic and hydraulic concepts. Dr. Watson is the co-author of a textbook on incised channel rehabilitation. Dr. Watson leads the geomorphology portions of the Waterways Experiment Station channel stabilization seminar. He is also involved in research to develop habitat design criteria in flood control channels. Dr. Watson also leads research into the development and testing of channel stabilization structures to maximize the natural reassertion capacity of streams.

William Taggart

William C. Taggart, P.E. is President and Founder of Taggart Engineering Associates, Inc. a consulting engineering firm specializing in water related engineering. Mr. Taggart has over 27 years of experience. He has a BSCE from the University of Denver in 1971 and an MSCE from Massachusetts Institute of Technology in 1972. Mr. Taggart has designed grade control and diversion structures for flood control and water supply on urban drainageways and various Western rivers including the South Platte, Arkansas, Colorado, Yellowstone, Snake, Gardner, and Kern. He has also designed dam modifications that include whitewater bypass structures for boating and fish passage. He was the author of the textbook, Urban Storm Drainage Management, published by Marcell Dekker, Inc., NY, 1982, and headed the investigation in Drop Structures in the Denver

Metropolitan Area prepared for the Urban Drainage and Flood Control District (UDFCD), December 1986. He is the primary author of the new "Structures" section of the Denver UDFCD's Criteria Manual.

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Rodney Wittler is an Hydraulic Engineer in the Water Resources Research Laboratory, Technical Service Center, US Bureau of Reclamation. Dr. Wittler has 15 years experience in hydraulic models, river mechanics, flow measurement, sediment analysis, instrumentation, bridge pier scour, computational methods, river restoration, riprap design, watershed planning, and engineering management. He is the Dam Foundation Erosion Study Team Leader. Dr. Wittler designed and implemented the stream restoration project on Muddy Creek near Great Falls, Montana and a cultural resources salvage project near Clark Canyon dam in southwestern Montana. Dr. Wittler represents Reclamation as a River Restoration specialist on the internal review team of the Trinity River Flow Evaluation for the Department of Interior.

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Martin J. Teal is a Project Manager and Senior Hydraulic Engineer with WEST Consultants in San Diego, California. His expertise is in the fields of hydraulics, hydrology and sediment transport. His experience includes numerous riverine and flood plain studies. Mr. Teal has designed grade control and energy dissipation structures for both public and private-sector clients. Mr. Teal is a registered Professional Engineer in Arizona and California and a certified Professional Hydrologist. Prior to joining WEST in 1993, he had worked for the US Army Corps of Engineers (Sacramento District) and Fluor-Daniel Chile. He earned his B.S. in Civil Engineering from the University of California, Berkeley, and his M.S. in Civil Engineering (Hydraulics) from the University of Iowa.

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Katherine J. Chase, P.E. is an associate of Taggart Engineering Associates, Inc.. Ms. Chase earned her BSCE from Texas A&M University in 1984 and then served as a Peace Corps volunteer. Her Peace Corps work took her to the mountains of Nepal where she designed irrigation and drinking water systems, suspended footbridges, culverts, and stream bank stabilization. Ms. Chase earned her MSCE from Colorado State University in 1992. Her thesis, Thresholds for Gravel and Cobble Motion, builds on her work as a graduate research assistant for the National Park Service. She has analyzed hydraulics for drop structures and performed construction observation for the Creekfront project on Cherry Creek in downtown Denver and worked on Cañon City's Arkansas River whitewater bypass.

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SITING, MONITORING, & MAINTENANCE FOR THE DESIGN OF GRADE CONTROL STRUCTURES

FINAL REPORT OF THE TASK COMMITTEE
EVALUATION AND DESIGN FOR GRADE CONTROL

INTRODUCTION

<I'll try and write something here, but will need editorial help from the rest of the committee. Much of what Bill has in the current draft may transfer here. Try and relate to stream restoration.>

The undisturbed alluvial stream is a rare and vanishing feature of nature. The impacts of urbanization, mining, agriculture, construction and other human activities have resulted in the need to darn and/or channelize many of the major waterways to convey flood runoff and maintain transportation lanes with a minimal impact on the adjacent properties. A stabilized river in natural regime is capable of conveying flood runoff with minimal changes in the channel bed and banks. Generally, the channel can convey a wide range of sediment laden flows with a minimal change in stage thereby adjusting the bed-form configuration and subsequent hydraulic roughness.

The damming and channelizing of the major waterways in conjunction with the changes in land use have significantly impacted the tributary stream Systems. The increase in runoff in conjunction with the reduction in sediment yield has resulted in extensive stream degradation. In many cases, the degrading stream bed has increased bank heights to a point of instability resulting in massive bank failure and a sediment infusion into the stream. For example, northern Mississippi streams that were documented in the 1950's with bank heights of 5 to 8 ft have degraded to bank heights of over 20 ft as documented in the early 1990's. The degradation of the channel has also resulted in the stream widths increasing 3 to 5 fold. In an attempt to stabilize these rapidly degrading streams, bank revetment and in-stream structures have been placed to stop bed degradation and bank failure. The long-term success of stream stabilization is yet to be determined.

One of key factors that directly influences our ability to stabilize the channel geometry is to establish an erosion-resistant strata for maintaining tile stream bed elevation. In nature a geologic hard point serves as a grade control feature. However, few geologic hard points occur where and when needed to stabilize the numerous degrading streams. Therefore, a man-made "hard point" is engineered. The engineered feature is routinely termed a grade control structure. The structure! serves as a temporary or non-permanent control point stabilizing the stream bed elevation. Once the bed elevation is relatively fixed, bank revetment and in-stream training structures may be placed to restrain stream lateral migration.

An abundance of alternative grade control features and structures have been developed for both rural and urban applications. For example, grade control may be attained through the design and construction of baffled apron drops, vertical drops, sloping rock drops! sloping grouted rock, concrete drops with and without low flow/trickle channels, driven sheet piles with rock apron, low drop structures (i.e. ARS low-drop) and high drop structures (i.e. SCS high drop). Many of these structures are currently considered not only for stream bed stabilization, It also for inclusion into stream restoration and rehabilitation projects. Unfortunately, available information on these structures pertains only to the structural design aspects. Little information exists on how to integrate grade control into a watershed on a system wide basis.

GRADE CONTROL ENGINEERING

<What are the needs or reasons for grade control? How to identify grade control applications. This section leads into the next, Terminology & Concepts, where we spell out the different types of structures to meet different applications.>

<The following section is from Nakato's International Literature Review. I think the final fate of this section is in the GRADE CONTROL STRUCTURE TYPES section later in the document.>

TERMINOLOGY & CONCEPTS

<This section starts with basic terminology, possibly from Abt's definitions, EM1601, and the annotated bibliographies. The section then goes into grade control concepts beginning with Hydraulic Control, then moving to Energy Dissipation (from UDFCD manual and Design of Small Canal Structures). This section is essentially the first half of the background leading to the second half on grade control structure types.>

DEFINITIONS

<Needs an introduction as well as a thorough edit.>

Grade Control Term	Definition
Bed slope:	The inclination of the channel bottom
Cellular Block Mattress:	Regularly cavitated interconnected concrete blocks placed directly on a stream bank or filter to prevent erosion. The cavities can permit bank drainage and the growth of either volunteer or planted vegetation when filter fabric is not used between the mattress and bank
Channel:	A natural or manmade waterway that continuously or periodically passes water
Channel cutoff:	A new relatively short channel (natural or artificial) formed when a stream cuts or is realigned through the neck of an oxbow (or horseshoe bend). A cutoff can also develop as successive high-water flows from a chute across the inside of a point bar
Check:	A structure built to regulate or raise the water level in a supply channel; an area of land irrigated by water confined by earth ridges
Check and drop:	A structure that combines the functions of both a CHECK and a DROP; the water level may be raised upstream of the structure and dropped on the downstream side
Check dam:	A structure placed bank to bank downstream from a headcut

Grade Control Term	Definition
Check Dam:	A low, temporary type of dam or barrier erected across a narrow watercourse to retard flow, minimizes erosion, and promotes the deposition of silt. It may be of stone mesh, with an apron of similar construction or of timber. The establishment of a thick growth of trees or bushes along the banks and bed of the channel also retards velocity and scour
Chevron weir:	A configuration of a grade control structure where the weir crest is angular in plan, usually with the apex of the weir crest upstream. Flow over the crest is perpendicular to the weir crest. Convergence of flow as it passes the weir is not great enough to cause reverse flow downstream of the weir. However, convergence is great enough to concentrate flow toward the middle of the channel and reattachment of the flow to banks downstream of a weir
Cut off:	A channel cut across the neck of a bend
Cutbank:	The outside bank of a bend, often eroding and across the stream from a point bar
Deadman:	An anchor buried in the ground to hold wires or ropes for winches or for holding bank protective works such as stone mesh sausages, logs, etc. Maybe constructed of heavy timber or concrete and well buried.
Debris dam:	A dam or barrier erected across a stream valley for retention of debris, driftwood, gravel, and silt
Dike (groin, spur, jetty, deflector):	A structure designed (1) to reduce the water velocity as streamflow passes through the dike so that sediment deposition occurs instead of erosion (permeable dike) or (2) to deflect erosive currents away from the stream bank (impermeable dike)
Discharge:	The volume of water passing through a channel during a given time, usually measured in cubic feet per second
Drop structure:	A structure built in a stream channel or gully to decrease the velocity of flow by lowering the slope or grade; the drop in velocity promotes the deposition of silt and prevents excessive channel scour. See Check Dam
Drop:	A type of GCS where the dissipation of excess head occurs primarily on the structural elements
Drop:	A type of GCS where the dissipation of excess head occurs in the area down steam of the structural element

Grade Control Term	Definition
Erosion:	In the general sense, the wearing away of the land by wind and water. As used herein, the removal of soil particles from a bank slope primarily due to water action
Fine particles(or Fines):	Silt and clay particles
Full channel width grade control:	A classification of grade control structures where the structural elements span the entire width of the stream at all design flows
Grade Control Structure (GCS):	A hydraulic structure for dissipating excess head in the stream wise direction or maintaining the elevation of the existing stream bottom
Grade Control Structure:	Structure placed bank to bank across a stream across a stream channel (usually with its central axis perpendicular to flow) for the purpose of controlling bed slope and preventing scour or head cutting
Grade-stabilizing structure:	A barrier of dam erected across a stream or gully to stabilize the grade and thus prevent lowering of the channel or further head cutting
Gully erosion:	See GULLY DRAINAGE
Gully stabilization (or waterway stabilization structure):	A structure designed to prevent or minimize bed scour or other erosion in a channel or gully. The structure does not provide floodwater storage
Gully drainage (or gully erosion):	Drainage that is concentrated in gutters or furrows. Gullying leads to rapid drainage and erosion of valuable soil may also be rapid if vegetation is sparse or has been removed.
Gully dam:	A small FARM DAM constructed across a drainage depression or gully for the purpose of impounding water for irrigation or other farm purposes
Head Cutting:	The action of an upstream moving waterfall or locally steep channel bottom with rapidly flowing water through an otherwise placid stream. These conditions often indicate that a readjustment of a stream's discharge and sediment load characteristics is taking place
High profile:	A classification of grade control structures where the flow super critical at the crest or on the ramp for the entire range of design discharges
Inter-structural water surface gradient:	The gradient of the water surface between successive grade control structures

Grade Control Term	Definition
Low profile:	A classification of grade control structures where the flow is sub critical or critical at the crest or on the ramp for the entire range of design discharges
Medium profile:	A classification of grade control structures where the flow transitions from sub critical or critical to super critical at the crest or on the ramp for any portion of the range of design discharge
Partial channel width grade control:	A classification of grade control structures where the structural elements do not span the entire width of the stream at any design flow
Piping:	Flow of groundwater through subsurface conduits in the bank
Point bar:	The bank in a bend that has built up due to sediment deposition
Preservation:	Maintaining a stream system in its current state (Create a static equilibrium)
Ramp:	A type of GCS where the dissipation of excess head occurs primarily on the structural elements
Reach:	A section of a stream's length
Reformation:	Putting a stream system into a new form, similar to, but different from the original state (And God created the earth......)
Rehabilitation:	Putting a stream system into its current maximum state of efficiency (Making the most of current situation)
Reinforced-earth bulkhead:	A retention structure fabricated by placing vertical panels (e.g., precast concrete) adjacent to an eroding stream bank and attaching bars or strips to the panels. The bars or strips are then run perpendicular to the panels into the Streambank. Compacted backfill material is placed in lifts as the bars or strips are laid to strengthen the bank
Restoration:	Putting a stream system into a former or original state (Back the way it was)
Revetment:	A facing of stone, bags, blocks, pavement, etc., used to protect a bank against erosion
Scour:	The erosive action of flowing water in streams that removes and carries away material from the bed and banks
Sediment:	Soil particles that have been transported away from their natural location by wind or water action

Grade Control Term	Definition
Sill:	A type of GCS with no dissipation of excess head, and the base level of the stream bed is maintained at the current elevation upstream of the structure
Stabilized grade:	That grade or slope of a stream or channel along which neither erosion nor silting occurs
Step concept:	The step concept is the concept of the relationship between individual grade control structures. The tailwater of a downstream structure, increasing in elevation in the upstream direction at the Inter-structural water surface gradient, must back onto the toe of an upstream structure, or be above the invert of the upstream structure
Streambank protection works:	Structure(s) placed in or near a distressed Stream bank to control bank erosion or to prevent failure
Streambank failure:	Collapse or slippage of large mass of bank material into the channel
Streambank:	The side slopes of a channel between where the streamflow in normally confined
Streambank erosion:	Removal of soil particles from a bank slope primarily due to water action. Climatic conditions, ice and debris, chemical reactions, and changes in land and stream use may also lead to bank erosion.
Streamflow:	The movement of water through a channel
Thalweg offset barbs (Wittler):	Otherwise called a jetty, bendway weir, toe dike, or barb. Primarily to move the thalweg away from the bank, always in a series of two or more individual barbs
Thalweg directional barbs (Reichmuth):	Otherwise called a jetty, bendway weir, toe dike, or barb. Primarily to redirect or turn the thalweg away from the bank, usually functioning alone
Toe:	The break in slope at the foot of a bank where the bank meets the bed
Toe bank:	The break in slope between the bank and the surrounding terrain
Toe scour:	Local scour in the bed that occurs immediately downstream of a grade control structure due to excess energy remaining in the flow
Velocity (of water in a stream):	The distance that water can travel in a given direction during an interval of time

Grade Control Term	Definition
Vortex weir:	A configuration of a grade control structure where the weir crest is not linear in plan, but arches (curvilinear) or bends (angular) in either the upstream or downstream direction. Flow over the crest is perpendicular to weir crest. Convergence or divergence of the flow as is passes the weir causes reverse flow downstream of weir along both banks

HYDRAULIC CONTROL

<from Henderson? Taggart?>

ENERGY DISSIPATION

<UDFCD manual and Design of Small Canal Structures, and Taggart>

GRADE CONTROL STRUCTURE TYPES

<I don't want this section to get bogged down in details. I envision many photo's, fewer drawings, and a small to medium number of different structure type descriptions. Emphasize references for design details.>

<There is a great deal of information from Taggart's draft that I have yet to insert. Here is where the bulk will go. I have reproduced the decision flow chart for selecting structure types from the UDFCD. I will work on inserting much of Taggart's stuff and other UDFCD stuff in this section.>

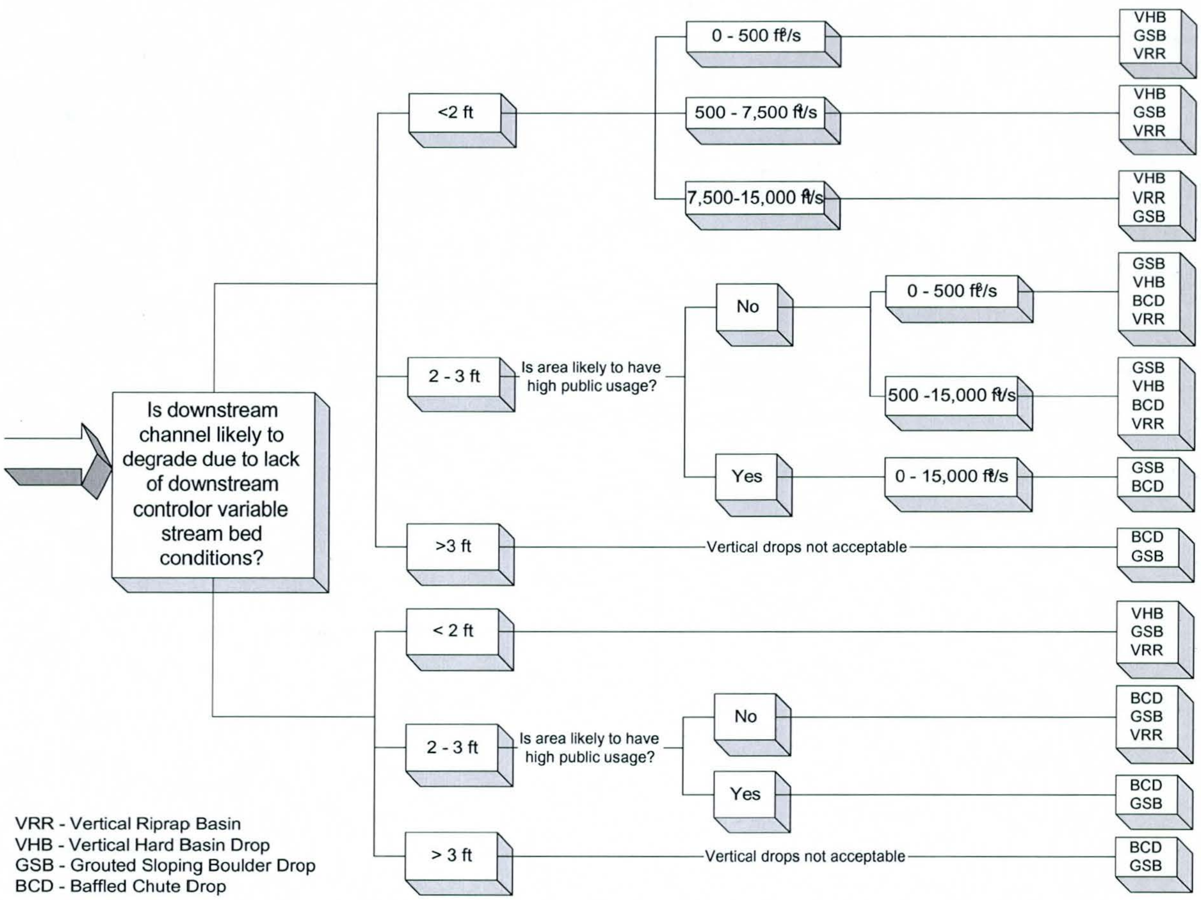


Figure 1. UDFCD Drop Structure Selection Chart.

A REVIEW OF INTERNATIONAL LITERATURE OF DESIGN PRACTICE AND EXPERIENCE WITH LOW-HEAD ALLUVIAL-CHANNEL GRADE-CONTROL DROP STRUCTURES

I. INTRODUCTION

Undisturbed alluvial streams are an endangered species in the United States, and, indeed, in most other parts of the world. Damming; diversion of part of the water flow but not the corresponding sediment load; imposition of increased sediment loads on rivers from mining, agricultural, construction, and other human activities; and flood-control measures, all cause major disruptions of the natural regime of a river. A river in natural regime is endowed with only the flow it receives from the watershed, with minimal changes in channel geometry. Streams in regime generally operate in a flow-sediment domain which enables them to convey wide ranges of flow with minimal changes in stage, by radically changing their bed-form configuration and attendant hydraulic roughness.

Major disruption of the natural regimes of large rivers, principally by damming and channelizing, has received extensive attention in recent years. However, relatively scant attention has been directed toward the responses of smaller streams to flood-control measures. The latter problem has become steadily more important as increasingly larger areas have become urbanized. The increased water runoff and reduced sediment yield from urban areas often produce rapid, extensive degradation of streams, which is further aggravated by channel narrowing resulting from diking or other encroachment on streams' overbank flow areas, and, in too many cases, even on their primary channels. The problem of small-stream degradation is by no means unique to the United States. Indeed, it is, no doubt, accurate to state that the problem is more acute in regions which are more densely populated, such as Japan, China, and many parts of Europe. In an effort to develop low-cost means of stabilizing degrading small streams, several different types of low-head drop structures, some of them quite novel, have been developed. The particular features of these depend strongly on the characteristics of the streams in which they are installed, and social attitudes toward man-made structures. Particularly in Europe, the latter aspect became so important in the last decade that many low-head drop structures are constructed with natural boulders to maintain the natural beauty of streams.

The principal objective of this investigation was to survey the international literature and to canvass the principal hydraulics design and testing organizations around the world for current information on the different types of low-head drop structures they have developed and installed; and reviewing their experience with these structures. The supplemental objective was to review the applicability of existing analytical models and so-called "kinematic analysis" to small streams which are grade-controlled with single or cascaded low-head drop structures to obtain rapid yet reasonably accurate predictions of the equilibrium bed profiles of the streams. A list of references reviewed in this study is given in Appendix I, and the names and organizations contacted for references during the study is listed in Appendix II.

II. SCOUR DEPTH BELOW DROP STRUCTURES

A. SCOUR DEPTH BELOW VERTICAL DROP STRUCTURES

Empirical formulas to predict scour depth below low-head drop structures with noncohesive bed materials have been compiled. Formulas which are considered to be useful for engineering purposes are listed in chronological order as follows:

1. Schoklitsch (1935, Reference: Whittaker and Schleiss 1984). Figure 2 defines the equation terms.

$$D_s = 4.75 \frac{h^{0.2} q^{0.57}}{d_{90}^{0.32}} \tag{1}$$

D_s - Scour depth below downstream water surface (m)
h - Drop height (m)
q - Unit discharge ($m^3/s/m$)
d_{90} - Bed material size for which 90% of the sample is finer (mm)

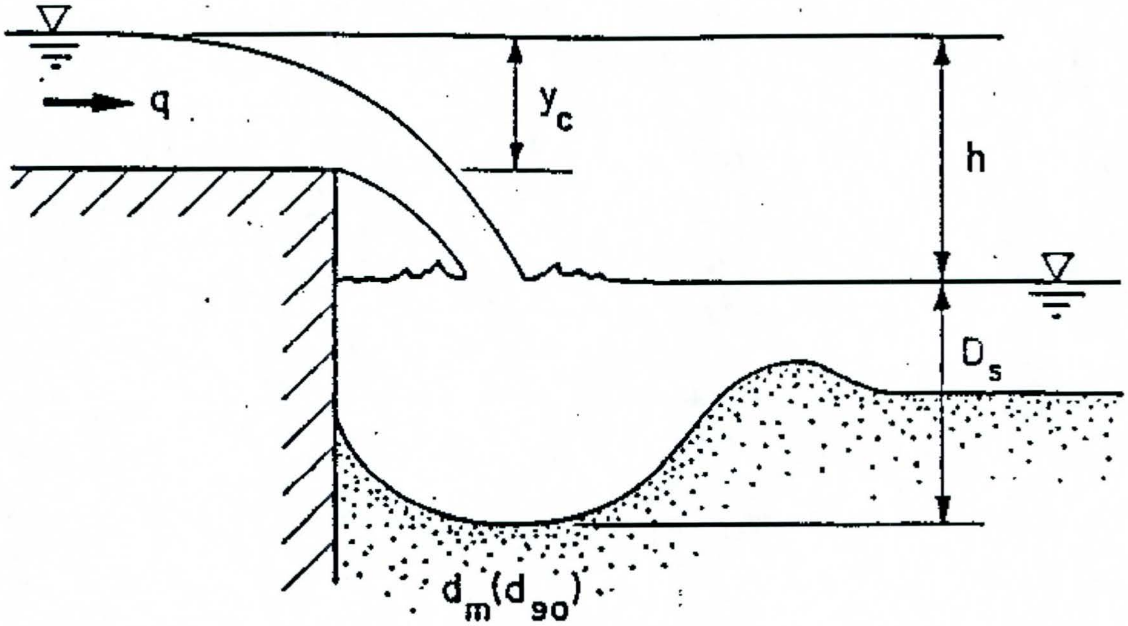


Figure 2. Definition sketch of scour below vertical drop structure.

2. Veronese (1937, Reference: Whittaker and Schleiss 1984).

$$D_s = 3.68\frac{h^{0.225}q^{0.54}}{d_m^{0.32}} \tag{2}$$

d_m - Median bed-material size (mm)

3. Kotoulas (1967, Reference: Whittaker and Schleiss 1984).

$$D_s = 0.78\frac{h^{0.35}q^{0.7}}{d_{90}^{0.4}} \tag{3}$$

d_{90} - Bed material size for which 90% of the sample is finer (m)

4. Bisaz and Tschopp 1972.

$$D_s = 2.76h^{0.25}q^{0.5} - 7.22d_{90} \tag{4}$$

d_{90} - Bed material size for which 90% of the sample is finer (m)

5. Aguirre Pe et al. 1980.

$$D_s = 0.864\frac{T^{0.29}(Q/T)^{0.42}h^{0.3}}{d_m^{0.21}} \tag{5}$$

T - Water surface width of the trapezoidal weir (m)
Q - Total discharge (m^3/s)
d_m - Median bed-material size (m)

Because T becomes equal to bed width for a rectangular weir, equation 5 does not seem to be correct. Scour depth should not depend on river width.

6. Laursen and Flick 1983, Laursen et al. 1986.

$$\frac{D_s}{y_c} = 8V_w^{0.75} - \frac{(6+V_w)}{(1+2h/y_c)^{0.5}} \tag{6}$$

y_c - Critical depth (m)
V_w - V_c/w
V_c - Critical flow velocity (m/s)
w - Particle fall velocity (m/s)

In order to evaluate these empirical formulas except equation 5, scour depth, D_s, was estimated for each case as a function of bed-material size, d, assuming that q = 0.011 m²/s and h = 0.19 m. It was also assumed for simplicity that $d_{90} = 1.5d_m$. Figure 3 shows a comparison of the equations. There are large discrepancies for particle diameters smaller than about 0.5 mm. However, their predictions converge as the particle size increases. Note that Hayashi 1974 also analyzed scour depth below a vertical drop as a function of the median bed-material size, unit discharge, and jet-flow velocity and jet-attack angle at the downstream water surface.

B. SCOUR DEPTH BELOW SLOPING DROP STRUCTURES.

Three important contributions on the subject of scour depth have been reviewed, including one contribution on cobble-lined drop structures.

1. Laursen and Flick 1983, Laursen 1984, Laursen et al. 1986.

$$\frac{D_s}{y_c} = 4\left(\frac{y_c}{d_m}\right)^{0.2} - 3\left(\frac{d_{rr}}{y_c}\right)^{0.1} \tag{7}$$

d_{rr} - Stone size of sloping drop structure
d_m - Median bed-material size

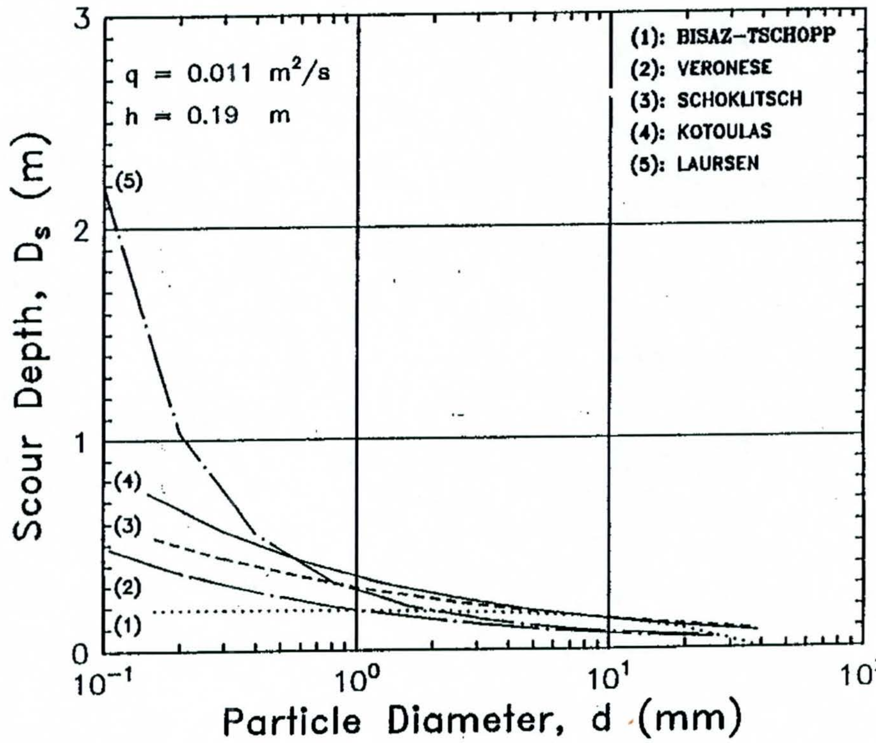


Figure 3. Scour depth prediction by different formulas for vertical drop structure.

Figure 4 defines these variables. Laursen et al. found that d_{rr} can be approximated by $2.8y_c$. Sloping sills were found to be stable when y_c was smaller than $0.8y_c$ for a sill slope of 25% (i.e., 4H:lV) and a range of between 0.06 and 0.4 feet.

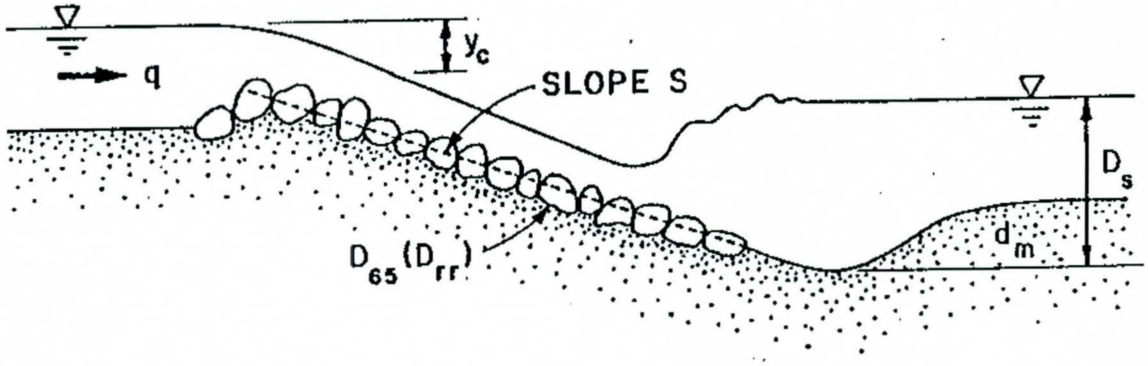


Figure 4. Definition sketch of a sloping drop structure.

2. Whittaker and Jaggi 1986

$$\frac{D_s}{y_c} = 1.05S^{-0.83}\left(\frac{D_{65}}{y_c}\right) - 7.13\left(\frac{d_m}{y_c}\right) \tag{8}$$

S - Slope of sloping sill

d_{65} - Bed material size for which 65% of the sample is finer

Note that equation 8 has a parameter of slope, S, that Laursen's formula does not include. In order to show general characteristics of equation 8, normalized scour depth D_s/y_c was computed as a function of y_c/d for different values of d_{65}/y_c. Figure 5 shows their relationships for S = 10%, and Figure 6 for S = 20%.

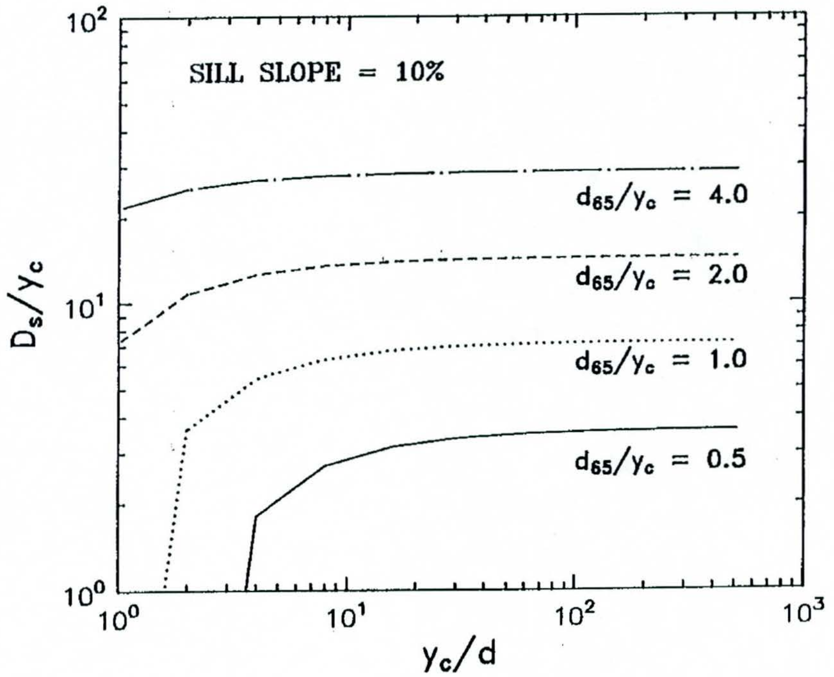


Figure 5. Scour-Depth prediction using the Whittaker and Jaggi 1986 formula for S=10%.

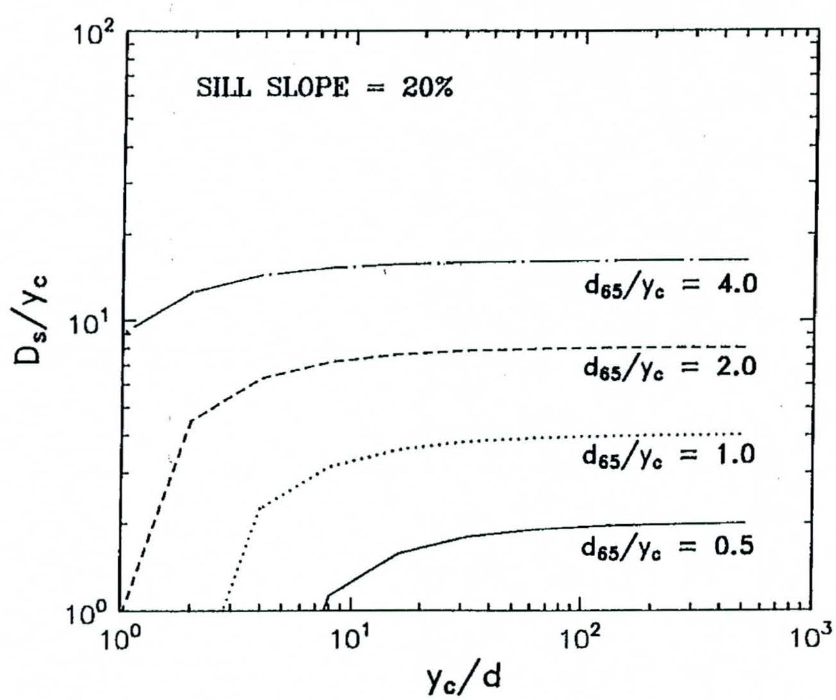


Figure 6. Scour-Depth prediction using the Whittaker and Jaggi 1986 formula for S=20%.

3. Smith and Murray 1975

Smith and Murray conducted laboratory flume tests for cobble-lined drop structures which comprise three basic elements, i.e., a weir at the top of the slope to reduce upstream flow velocity and prevent excessive draw down, a sloping cobble-lined section downstream from the weir, and a horizontal apron at the bottom. Three different sizes of cobbles, ranging from 15.3 mm to 23.5 mm) w ere tested for four different slopes of 4, 5, 6, and 7%. They found that the terminal flow depth, y, on the sloping section is a function of the ratio of the median bed material size and the slope.

$$y = 0.1\frac{d_m}{S} \tag{9}$$

It was also found that initial failure of the slope would occur when y exceeds $0.116d/S$, approximately, but the structure will stabilize after the initial failure, provided that a stone depth larger than $3d_m$ is available at the top of the slope, allowing a continued operation thereafter.

III. SEVERAL EXISTING DESIGN GUIDELINES

A. JAPANESE GUIDELINES.

The Japanese Ministry of Construction specifies general guidelines for designing low-head drop structures (Japanese Ministry of Construction 1976, Japanese Ministry of Construction 1985). Figure 7 illustrates each structural component discussed in detail in the references.

The maximum drop height is specified to be about 3.5 m; however, the guideline allows higher drops in the case of a very steep river section which would require too many drop structures The slope of the downstream face of the main concrete body is generally between OH:lV: and O.5H:lV; however, the face slope could be as large as lH:lV in order to suppress noise produced by falling water in urban areas. The general procedure for the stability analysis of the main drop structure is given for common forces including dead weight, earth pressure, hydrostatic pressure, uplift pressure, and inertia force due to earthquake; overturning; sliding; and compressive strength of the foundation.

The stilling basin shown in Figure 7 is a concrete structure connected to the main drop structure, and should be designed to withstand erosion and uplift forces. The stilling-basin length, L, is:

$$L = 0.6C_o W^{0.5} \tag{10}$$

L - Stilling basin length (m)
C_o - Seepage path coefficient

Silt and very fine sand	-	*18*
Fine sand	-	*15*
Coarse sand	-	*12*
Sand-Gravel mixture	-	*9*
Gravel	-	*4-5*

The guideline recommends that L be, in general, about 2 to 3 times the drop height. The thickness of the stilling basin, t, is:

$$t = F_s \frac{(U_{max} - h_2 W_w)}{(W_c - 1)} \tag{11}$$

F_s - Safety factor (generally 1.333)
U_{max} - Maximum uplift (t/m^2)

h_2 - Tail water depth
W_w - Specific weight of water
W_c - Specific weight of concrete (2.3-2.35 t/m^3)

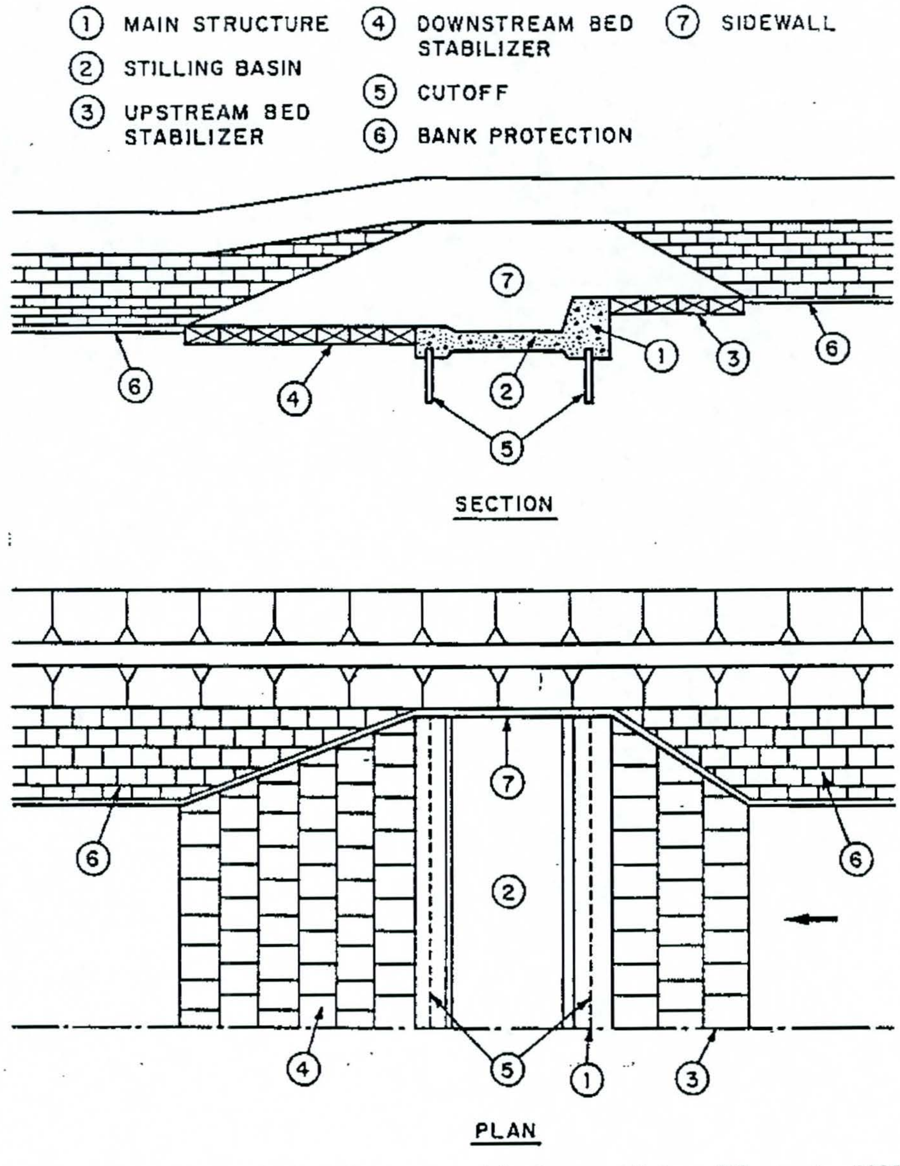


Figure 7. A typical Japanese low-head drop structure. (after Japanese Ministry of Construction 1985)

Figure 8 shows more detail of the concrete portion of the stilling basin/drop structure. Note that in both Figure 7 and Figure 8 the flow is right to left. Lane's weighted creep method (US Bureau of Reclamation, 1973) calculates cutoff depths. The guideline also specifies the length of downstream bed stabilizer, L_d. A bed stabilizer is also required upstream from the drop structure. The length is generally up to ½of L_d.

$$L_d = 0.67C_o(W - h_2)^{0.5} - L \tag{12}$$

L_d - Length of downstream stabilizer (m)
W - Drop height (m)

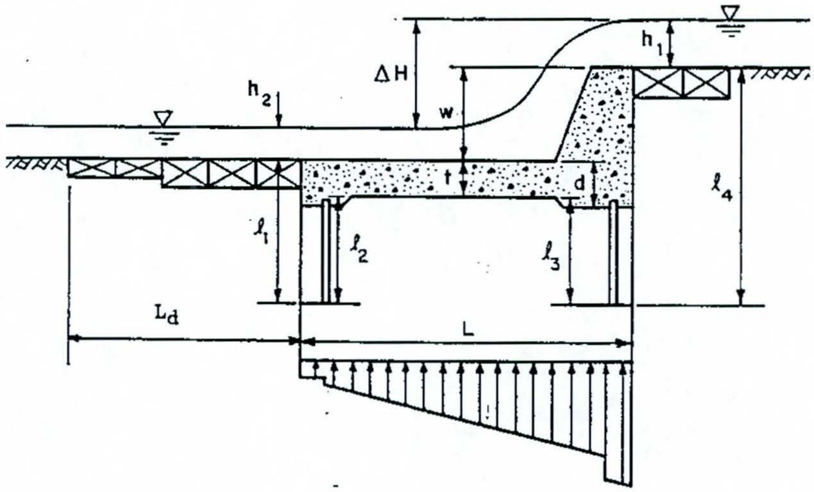


Figure 8. Definition sketch of typical Japanese low-head drop structure.

Ashida et al. 1975 describe general design procedures for low-head drop structures It is assumed that the brink depth is ah_c and the drop height is W. The lateral distance, L_d, traveled by a water particle, then, is a function of a and D.

$$\frac{L_d}{W} = \left(\frac{0.2^{0.5}}{a}\right)^{1/6}\left(1 + aD^{1/3}\right)^{1/2} \tag{13}$$

h_c - Critical depth
a - Brink-depth coefficient (0.215-0.715)
D - Drop number = $q^2/(gW)^3$
q - Unit discharge

Figure 9 is a definition sketch of this situation. When the upstream bed slope is mild, the coefficient, a, is approximately equal to 0.715. Figure 10 shows the functional relationship between a and F_r. When the slope is steep, it is a function of the upstream Froude number, F_r. Rand 1955 obtained an empirical equation similar to equation 13 for the mild slope.

$$\frac{L_d}{W} = 4.30D^{0.27} \tag{14}$$

Figure 11 shows the relationship between L_d/W and D which includes experimental data obtained by Ashida et al. As can be seen from Figure 11, L_d/W may be larger than 2.0. The stilling-basin flow depth, h_1, immediately after impingement is given by White 1943 as

$$\frac{h_1}{W} = \frac{2D^{0.5}}{\left[\left\{\left(\frac{a+1}{a^2}\right)D^{1/3}\right\}^{1/2} + \frac{D^{1/6}}{a}\right]} \tag{15}$$

While Rand 1955 proposed the following empirical formula:

$$\frac{h_1}{W} = 0.54D^{0.425} \tag{16}$$

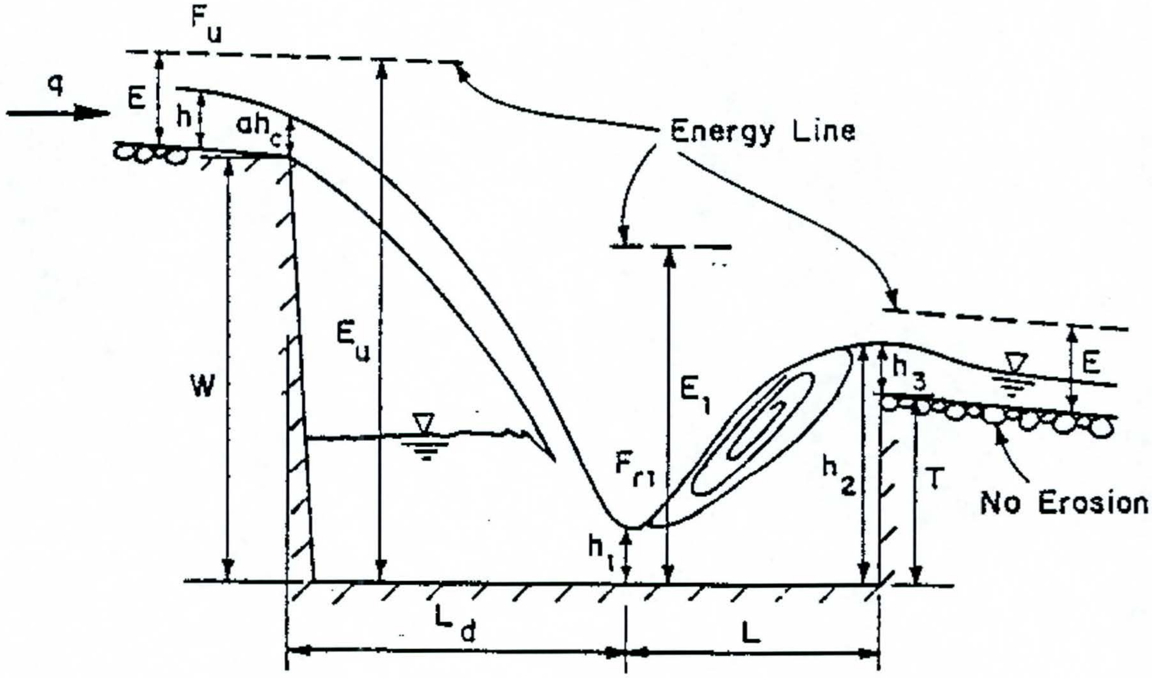


Figure 9. Definition sketch of low-head drop structure. (after Ashida et al. 1975)

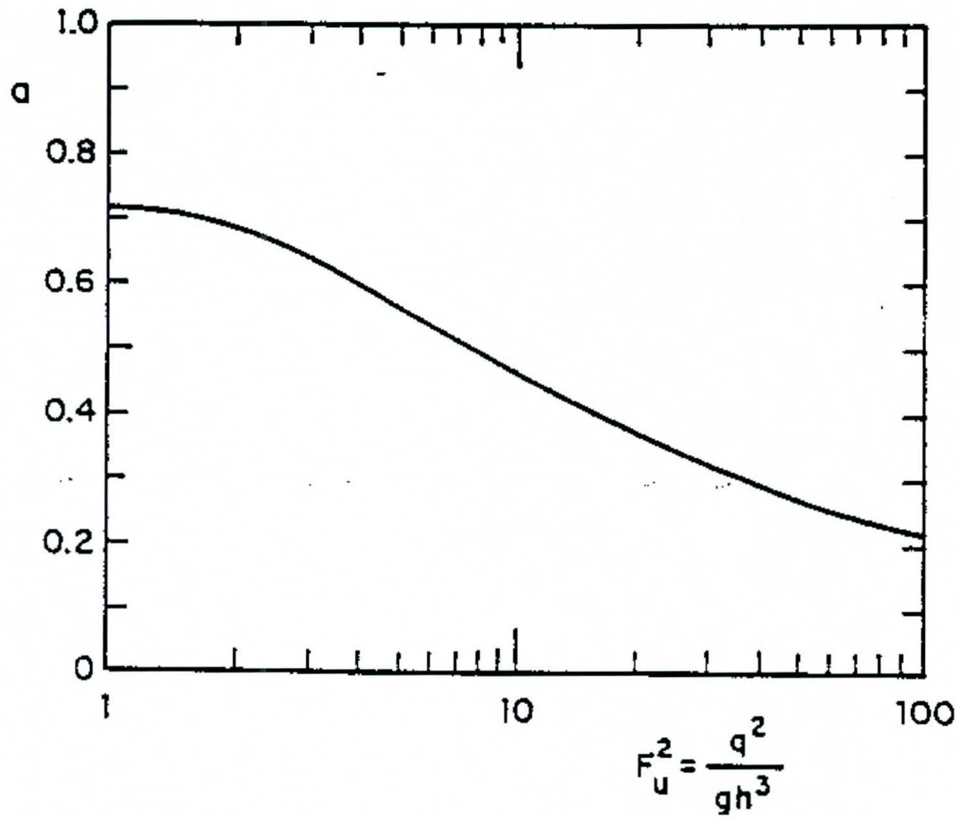


Figure 10. Relationship between F_r^2 and a (after Ashida et al. 1975)

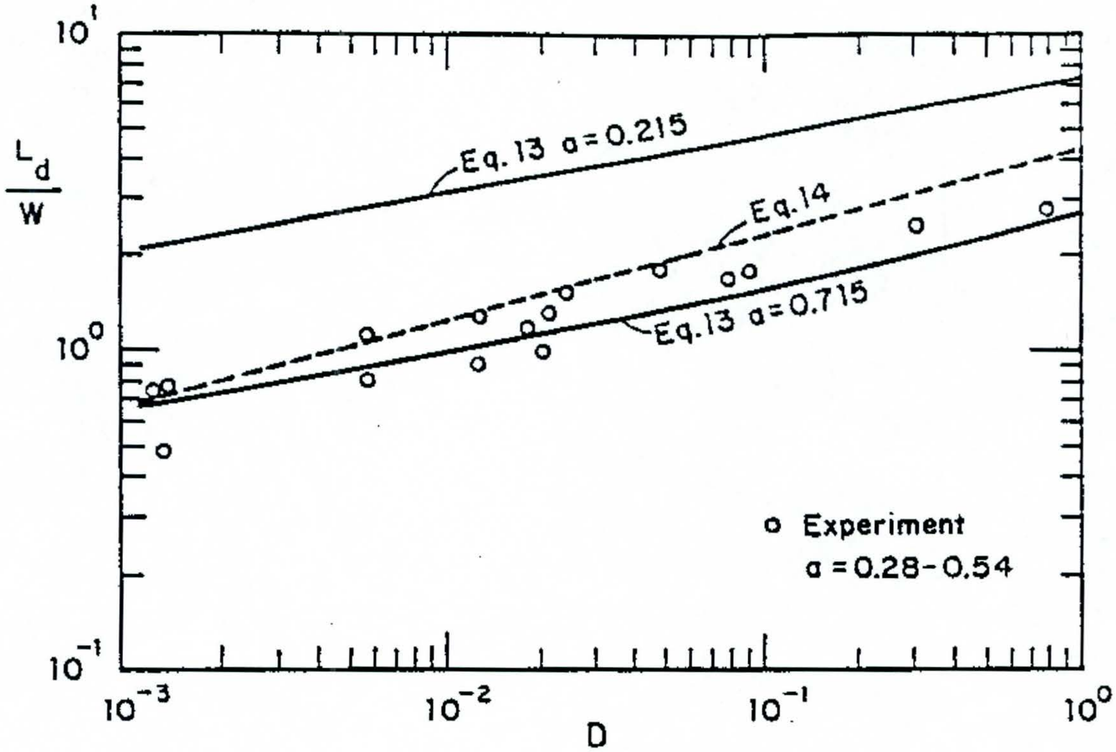


Figure 11. Relationship between D and L_d/W (after Ashida et al. 1975).

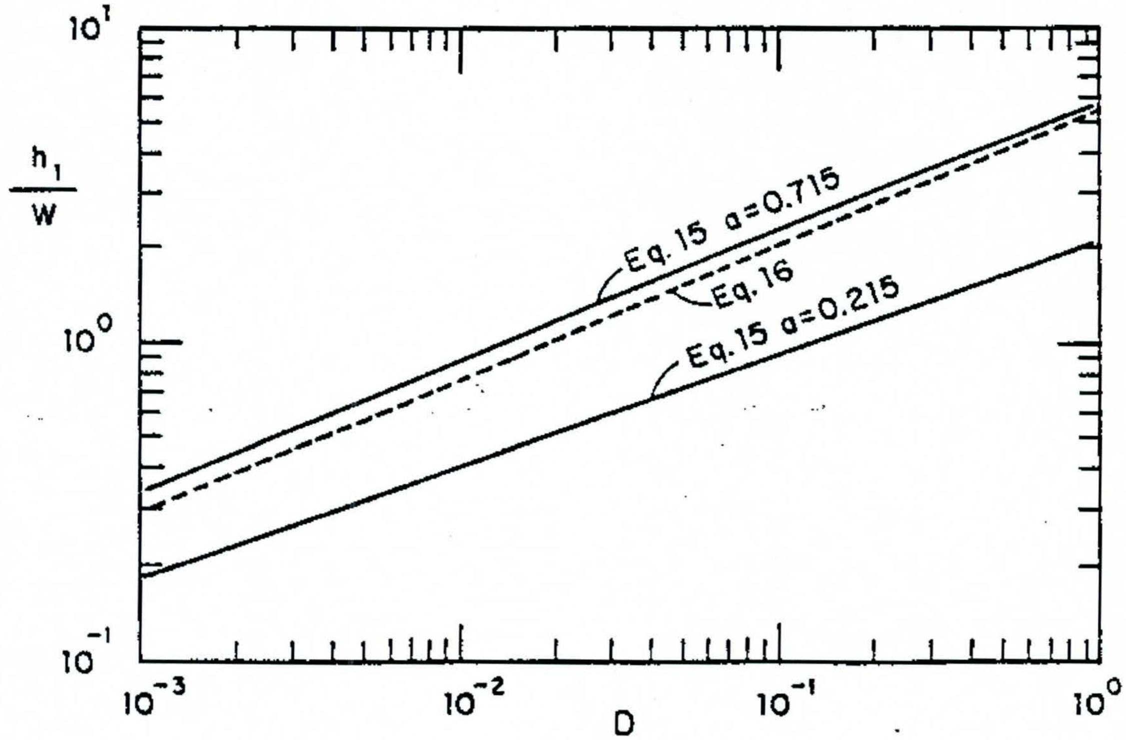


Figure 12. Relationship between D and h_1/W (after Ashida et al. 1975).

Functional relationships given by equations 15 and 16 are depicted in Figure 12. The empirical relationship obtained by Rand is seen in this figure to be very close to the theoretical curve for the mild slope.

Total energy at the brink depth, E_u, is given by

$$\frac{E_u}{W} = 1 + \left(\frac{a+1}{(2a)^2}\right) D^{1/3} \tag{17}$$

Total energy at the stilling basin, E_1, is given by

$$\frac{E_1}{W} = \frac{\left(2D^{1/2} + 0.125 D_1^{1/2} + \frac{D^{1/6}}{a}\right)^3}{\left(D_1^{1/2} + \frac{D^{1/6}}{a}\right)} \tag{18}$$

$$D_1 = 2 + \left(a + 1/a^2\right) D^{1/3}$$

Therefore, the relative energy loss due to the drop structure, $(E_u - E_1)/W$, can be obtained as a function of D and a, as Figure 13 shows. As can be seen in this figure, only approximately 0.5W-0.6W of energy is dissipated through the impingement for a range of D between 0.001 and 1. This means that the remaining energy must be dissipated by some other means. One way is to use sill blocks within the stilling basin so that energy is dissipated through hydraulic jump. The other method would be to provide a long stilling basin so that energy can be dissipated through bed friction.

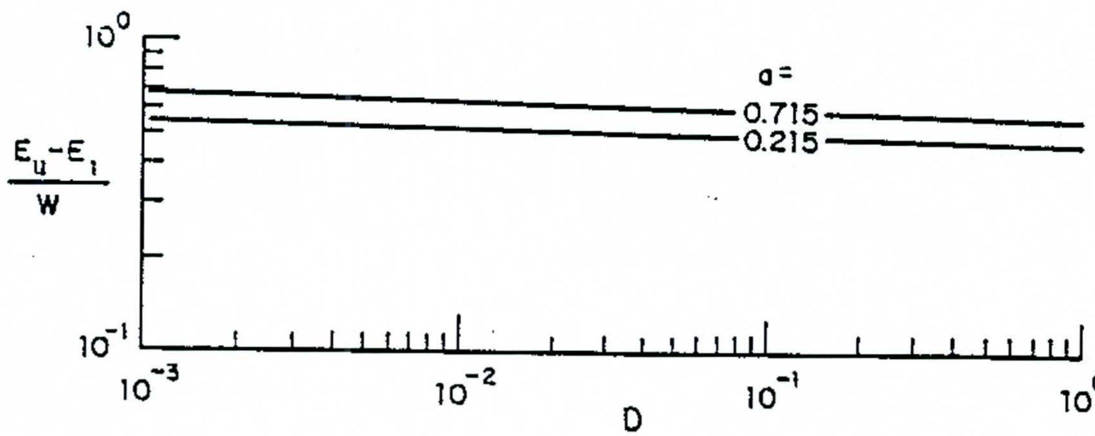


Figure 13. Relationship between D and $(E_u$-$E_1)/W$ (after Ashida et al. 1975).

In order to obtain satisfactory hydraulic jump within the stilling basin, it is necessary to maintain the upstream Froude number, Fr_1, somewhere between 4.5 and 9.0. Otherwise, rollers generally develop immediately downstream from the jump and propagate downstream. Therefore, there exists a limit in adopting a proper drop number, i.e., drop height, W, according to the definition of the drop number. Figure 14 shows conditions under which stable hydraulic jumps can be achieved (see Figure 9 for parameter definitions). As can be seen in this figure, a larger drop height (or smaller drop number) is required for a smaller upstream Froude number, Fr_u. For example, if Fr_u is less than 1.0 and q is about 10 m^2/s, the stable hydraulic jump would require that the drop height be far larger than 15 m according to Figure 14. This drop height is far larger than the limitation of 3.5 m specified by the Japanese Ministry of Construction (Japanese Ministry of Construction 1976, Japanese Ministry of Construction 1985). This limitation could be met if Fr_u is about 4.0. Another example is that for Fr_1 = 4.5 the downstream Froude number, Fr_2, must be about 0.32 in order to produce satisfactory hydraulic jump. This small Froude number is generally rare in natural flow conditions. Therefore, a sill whose height is T, as shown in Figure 9, becomes necessary. The sill height can be determined by Iwasaki's formula (Japan Society of Civil Engineers 1964):

$$\frac{T}{h_1} = \frac{\left[\left(1+2Fr_1^2\right)\left(1+8Fr_1^2\right)^{0.5}-1-5Fr_1^2\right]}{\left[1+4Fr_1^2-\left(1+8Fr_1^2\right)^{0.5}\right]} - \frac{3\left(Fr_1^2\right)^{1/3}}{2} \quad (19)$$

Using equation 19, T/W can be calculated to be about 0.11 for $Fr_1 = 4.5$ and $Fr_u < 1.0$. The length, L, required for satisfactory hydraulic jump (see Figure 9) is given by

$$L = 4.5h_2 \quad (20)$$

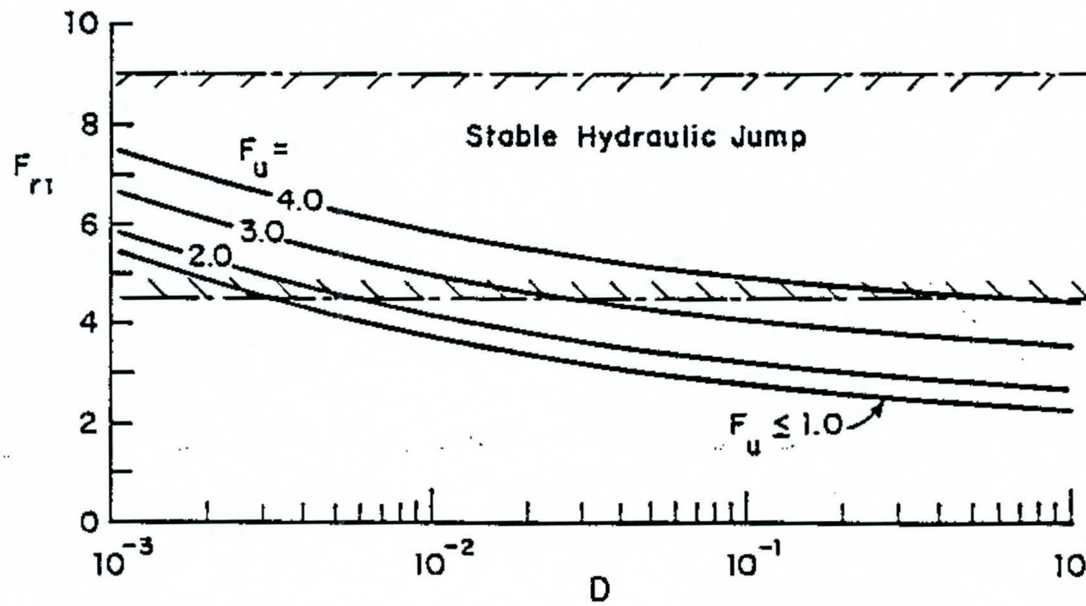


Figure 14. A stability diagram for stable hydraulic jump (after Ashida et al. 1975)

It should be noted that the use of hydraulic jump in dissipating energy generally requires a large drop height and sill structures; therefore, this type of method may not be advisable for a case in which downstream flow is supercritical. If the downstream river bed is stable under a supercritical condition, flow remains supercritical even in the stilling basin; therefore, artificial roughness may be needed within the stilling basin in order to maintain the stilling-basin flow depth larger than the uniform depth.

B. DENVER URBAN DRAINAGE AND FLOOD CONTROL DISTRICT GUIDELINES

A comprehensive review of general design guidelines and field experience for drop structures in the Denver Metropolitan area is reported by McLaughlin Water Engineers, Ltd.(MWE) (McLaughlin Water Engineers 1986, Taggart et al. 1987). The MWE report prepared for the Urban Drainage and Flood Control District (UDFCD) presents the most elaborate effort, to the author's knowledge, of reviewing various types of existing, low-head drop structures on grass-lined channels which are in the Denver Metropolitan area, and of assessing available references. MWE's study results and their views on various engineering aspects for different types of drop structures are summarized herein.

MWE evaluated six different types of drop structures, including baffled chute drops (BCD), vertical riprap drops (VRD), vertical hard-basin drops (VHD), sloping riprap drops (SRD), sloping grouted riprap drops (SGRD), and sloping hard-basin drops (SHD).

The general design guidelines for BCD are given by the U.S. Bureau of Reclamation (US Bureau of Reclamation, 1973, US Bureau of Reclamation 1978). A baffled chute is normally constructed on an excavated slope, 2H:lV or flatter, as shown in Figure 15. Backfill is generally placed over one or more rows of baffles to restore the original streambed elevation. The recommended design discharge per unit width ranges from 35 to 60 ft^3/s. Recommended baffle-pier height and allowable approach-flow velocity are shown as a function of the unit water discharge in Figure 16. Detailed design procedures are found in references (USBR, 1978 and MWE, 1986). MWE found that BCD's work excellently in grass-lined channels, except for cases under which heavy debris flows exist. BCD's seem to be best suited to shallow tailwater and variable bed conditions because the backfill allows the downstream channel bed to be scoured, while successive rows of exposed baffles would act then to prevent further scour development downstream. MWE strongly recommends incorporation of an 18- to 24-in. deep trickle channel into the BCD design. The trickle channel can be located between two baffles in the middle of the apron crest, as shown in Figure 17.

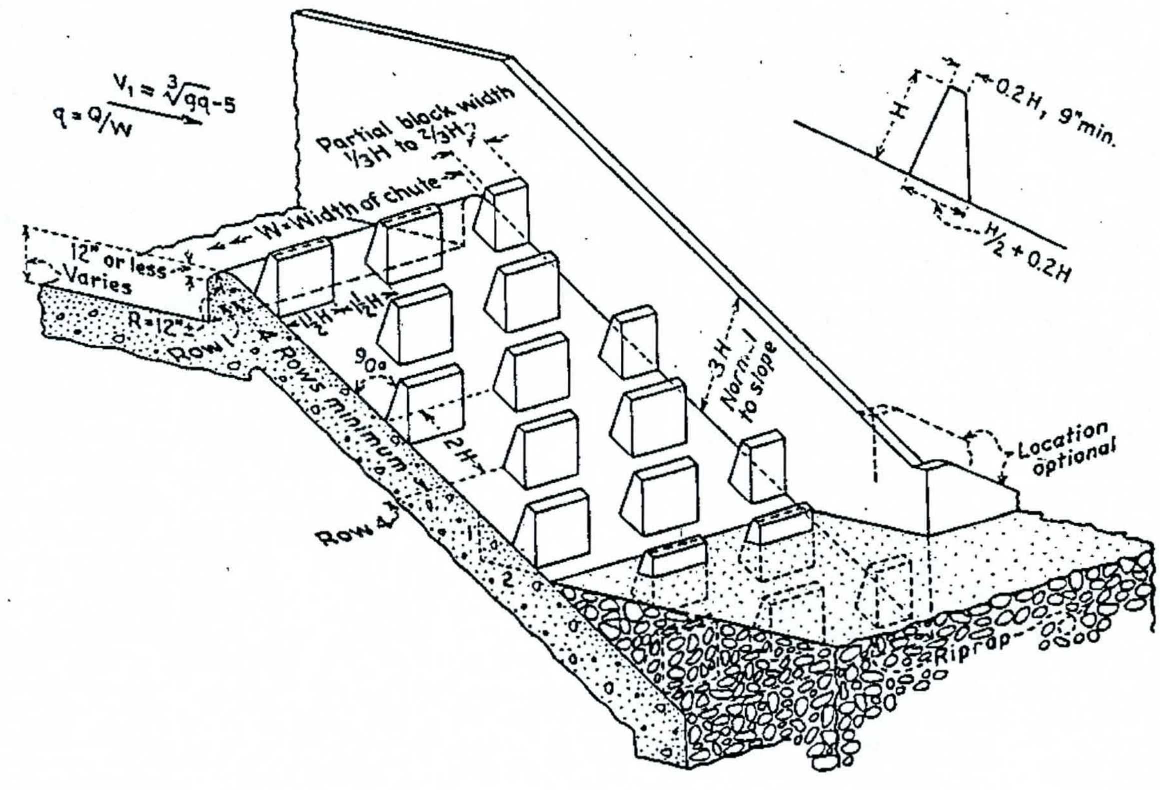


Figure 15. Basic components of baffled chute (after US Bureau of Reclamation 1978)

A vertical riprap drop (VRD) is designed to dissipate energy through plunging action of flow into a downstream pool. McLaughlin Water Engineers 1986 investigated five prototype cases and found that most of them seemed quite stable, indicating that they are better alternatives than the existing UDFCD sloping riprap drops. One reason cited is that the riprap material in VRD tends to settle in place, providing a deeper plunging pool for energy dissipation. MWE recommends that a trickle channel notch through the crest and a proper transition to the downstream trickle be provided so as to minimize aggradation upstream and local degradation downstream. The UDFCD uses a modified version of the design procedure developed by Smith 1985. MWE's design guideline for VRD with a trickle channel is shown in Figure 18.

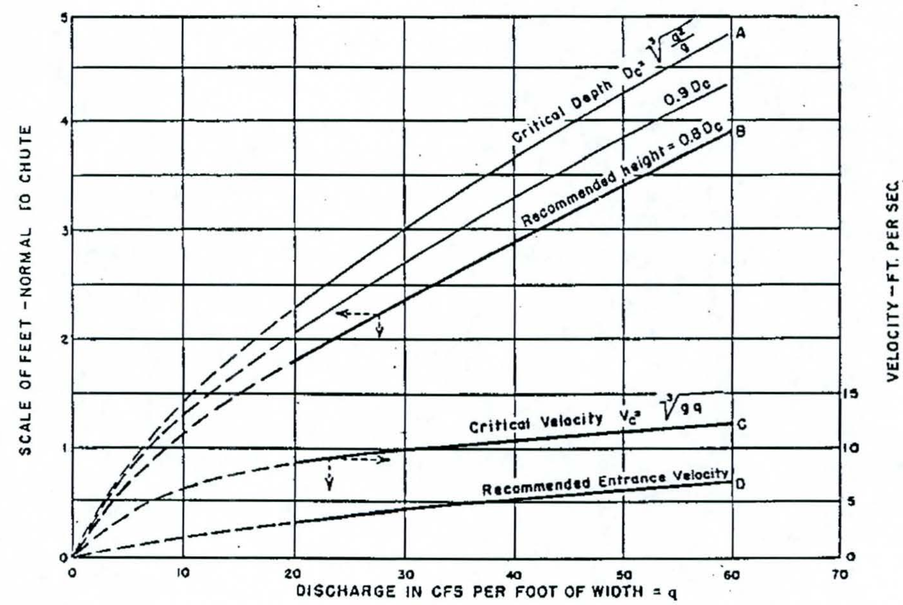


Figure 16. Recommended baffle pier height and allowable velocity)after US Bureau of Reclamation 1978)

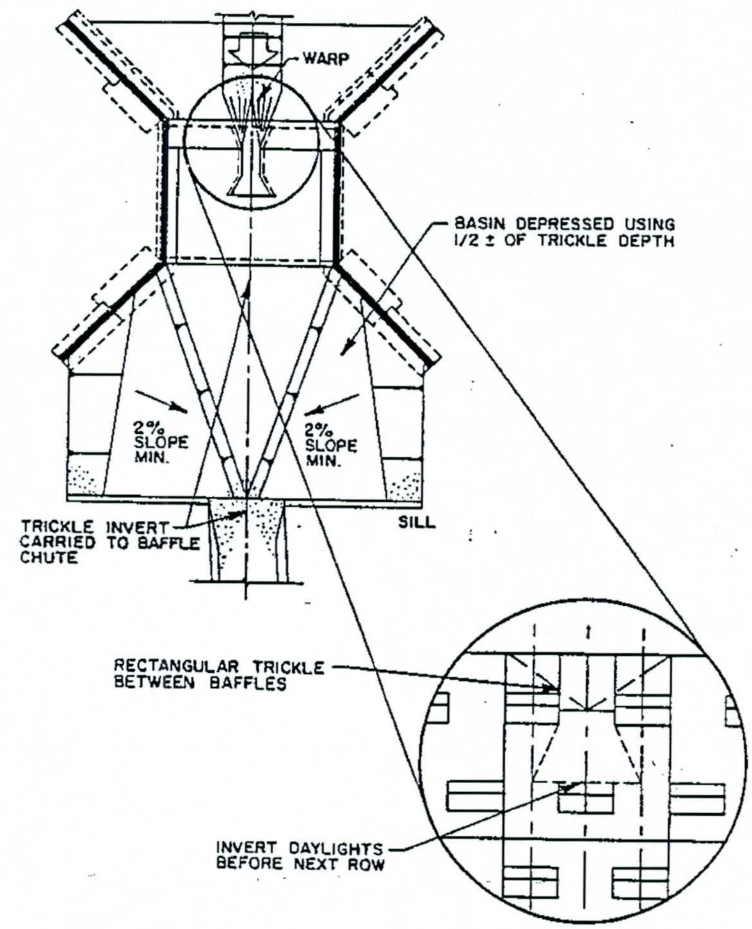


Figure 17. Details of baffled chute for trickle channel and hard stilling basin (after McLaughlin Water Engineers 1986).

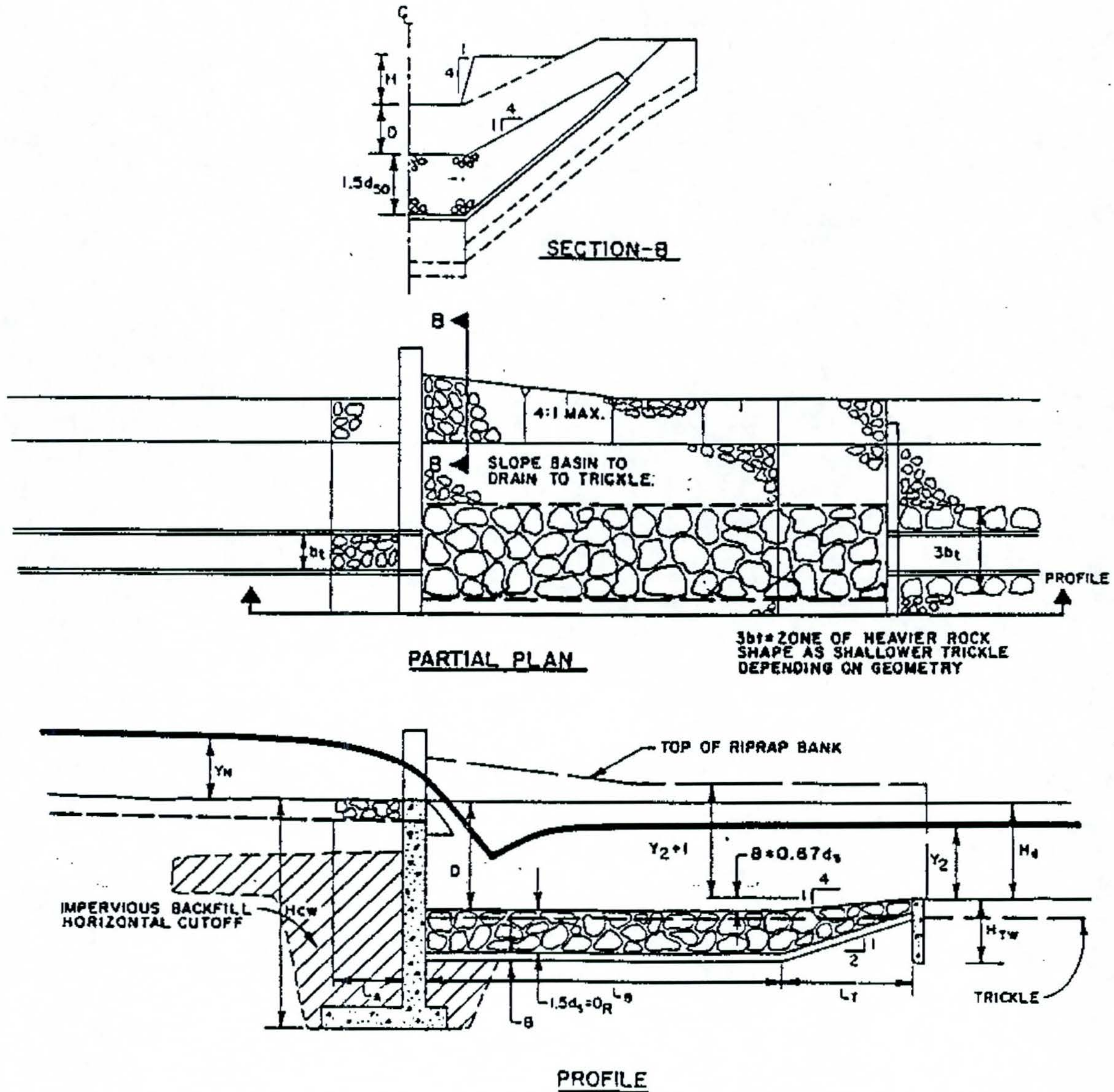


Figure 18. Details of vertical riprap drop (after McLaughlin Water Engineers 1986).

Performance of prototype, vertical hard-basin drops (VHD) in the UDFCD was evaluated by MWE McLaughlin Water Engineers 1986, which includes a design discharge varying from 220 ft^3/s to 3,000 ft^3/s. The basic design guidelines for VHD are given elsewhere, notably by Rand 1955 using a parameter called the drop number (see also Ashida et al. 1975; Chow 1959). Model investigations of vertical drops which connect trapezoidal channels are reported by Shih and Parsons 1967, and Fiuzad 1987. Theoretical and experimental investigations on flow characteristics at the base of the rectangular vertical drop structures are reported by Gill 1979, and El Khashab et al. 1987. Gill 1979 modified classical approaches proposed by Moore 1943 and Rand 1970 in the analysis of flow at the base of the vertical drop. Gill proposes modified expressions for flow depth in the pool below the falling jet and jet trajectory. His experimental results seem to support his modified approach. MWE concluded that VHD they evaluated seemed to be satisfactory. However, they recommend that further refinements be made with regard to trickle channels, better stilling-basin drainage, better upstream and downstream transition sections, and safety aspects to discourage people from approaching the dropwall. Figure 19 shows a VHD configuration with a trickle channel.

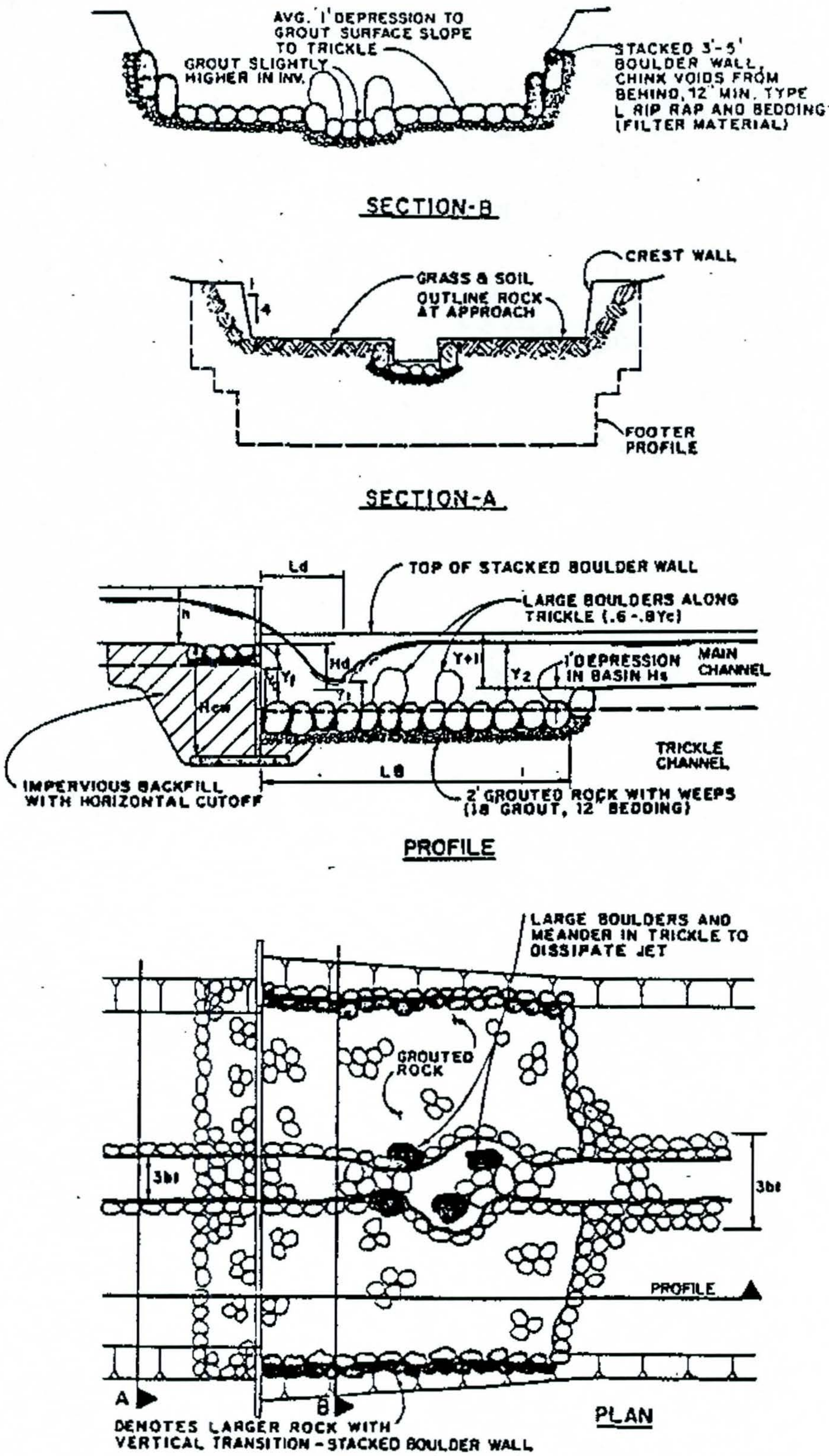


Figure 19. Details of vertical had basin drop (after McLaughlin Water Engineers 1986).

Many cases of sloping rock drops were surveyed by MWE which found many severe failures within the base width of the drop, and concluded that sloping drops with graded riprap should be used only in special cases in which extensive engineering and field quality control of graded riprap are available. The main reason seems to be due to poor quality control of rock gradations which is based on eyeball measurements of stockpiles. Placement of riprap at the site also tends to result in all larger rock in one area, because it nay be in the outside of the pile, and all smaller rock in another area. According to MWE's field quarry test of a 21-ton graded riprap sample, the rock was found to be short on larger sizes although the test team felt that gradation was reasonable using the eyeball method. For example, the actual spherically-equivalent size D65 for which 65% of the sample is finer by weight was only 19 in. for the specified design range between 22 in. and 26 in., approximately. MWE developed design guidelines for three different ranges of drops, i.e., 2-ft drops, 4-ft drops, and 4- to 6-ft drops, using a Shield's parameter of 0.091 and a safety factor of 1.5. Their charts provide us with required -median riprap diameters for a range of unit-width discharge between about 5 and 75 $ft^3/s/ft$ for total flows less than 500 ft^3/s. Figure 20 shows the plan and section of sloping riprap drop recommended by MWE.

C. GERMAN GUIDELINES

von Hausler 1976 presents his elaborate work on drop structures His paper describes basic hydraulic characteristics of vertical drops and steep-slope drops with stilling basins. He distinguishes a drop without an upstream damming effect (Absturz) from a dammed-up drop (Stutzschwelle) for which a weir equation must be used in the head-discharge relationship. Possible interactions between flow and drop structure in a cascade-type, multiple drop structure system are illustrated in Figure 21. In an ideal case, hydraulic jump takes place within each stilling basin, requiring a minimum length of downstream stabilizer, as can be seen in Figure 21-(a). If the drop structure is not designed appropriately, for example, in a case where tail-water depth is smaller than the sequent depth of flow at the downstream end of the drop, hydraulic jump moves downstream, thereby requiring an extended stretch of channel-bed stabilizer, as is illustrated in Figure 21-(b). Undesirable cases beyond the second case described above are also illustrated in Figure 21-(c) and (d) in which no stable flow-regime changes can be achieved by means of drop structures .

von Hausler 1976 discusses hydraulic jump below a drop with special emphasis on the tail-water influence on drop height. He introduces a parameter, S, called a degree of tail-water influence, which is typically between 1.1 and 1.5. The parameter, S, is defined as a ratio of the tail-water depth, t_u, to the sequent depth, t_2 of t_1 (see Figure 22). Hausler presents his concept in charts, as shown in Figure 23, in which a drop height, h_w, normalized by a critical velocity, t_{gr}, is shown as functions of several variables, including t_u/t_{gr}, upstream and downstream Froude numbers, and S. In this figure, a large discrepancy between the conventional German approach (DIN) and his approach in estimating h_w/t_{gr} can be seen. In his example, the unit width discharge is 15.29 $m^3/s/m$ (i.e., critical velocity = 2.88 m/s); the upstream bed elevation is at 344.8 m; the downstream bed elevation is at 343.0 m; and the tail-water elevation is at 350.0 m. Because the tail-water depth is 7.0 m, t_u/t_g = 2.432, which gives us a value of 1.51 in h_w/t_{gr} (or h_w = 4.35 m) according to the DIN formula, resulting in a dammed-up form (Stutzschwelle) of drop structure because the crown elevation in this case is 347.35 m which is higher than the upstream bed elevation of 344.80 m. Therefore, the DIN's formula which is applicable to conventional drops (Absturz) cannot be used. If S is assumed to be 1.1, as in Hausler's approach, h_w/t_{gr} is 2.87 (or h_w = 8.25 m), as can be seen in Figure 23. His approach seems to provide more reasonable results than the one using the DIN formula. Standard design sections of vertical and sloping drop structures are presented in Figure 24 and Figure 25. Note that Knauss 1983 also discusses the hydraulic design and structural arrangement of round and broad-crested weirs.

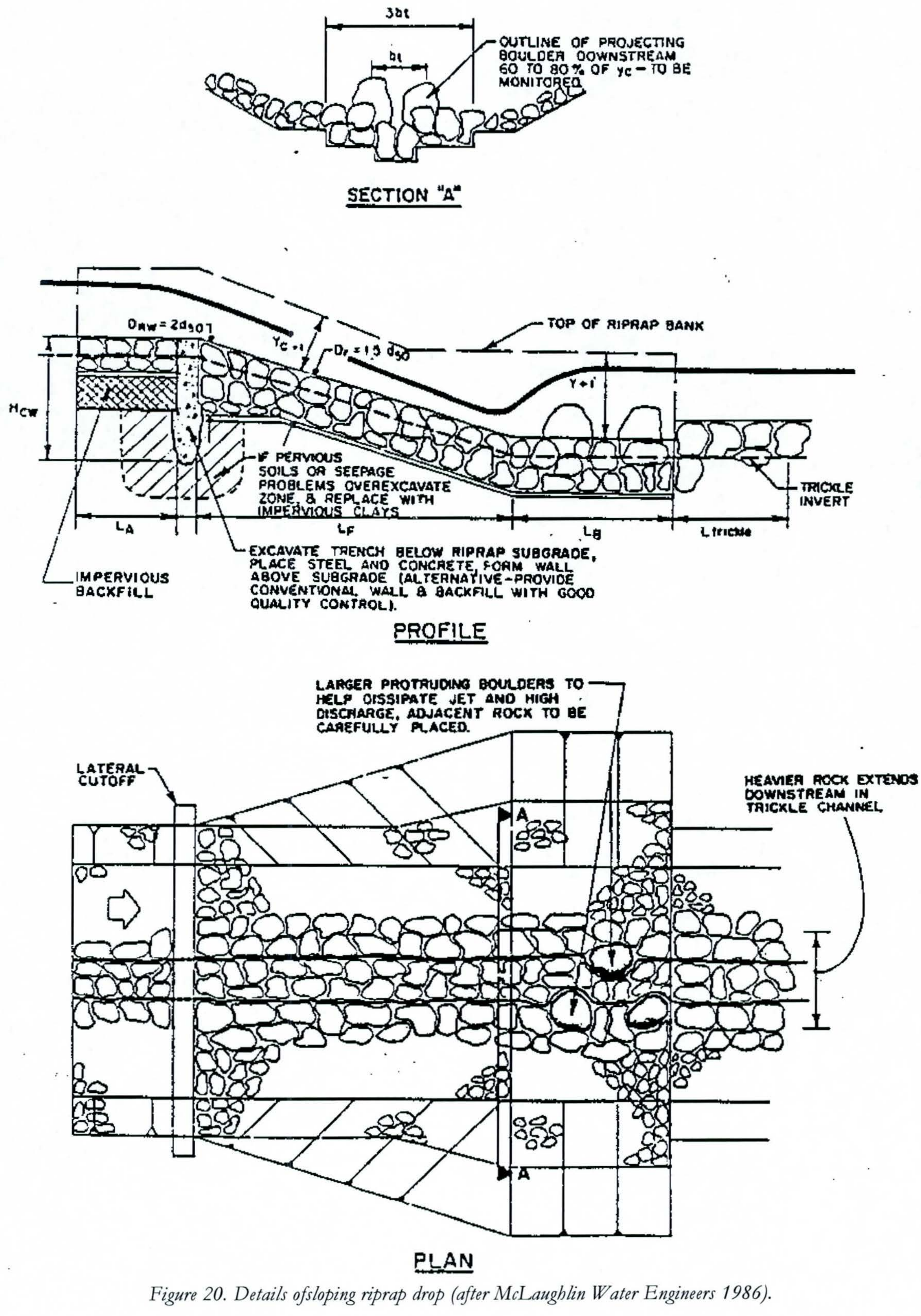


Figure 20. Details ofsloping riprap drop (after McLaughlin Water Engineers 1986).

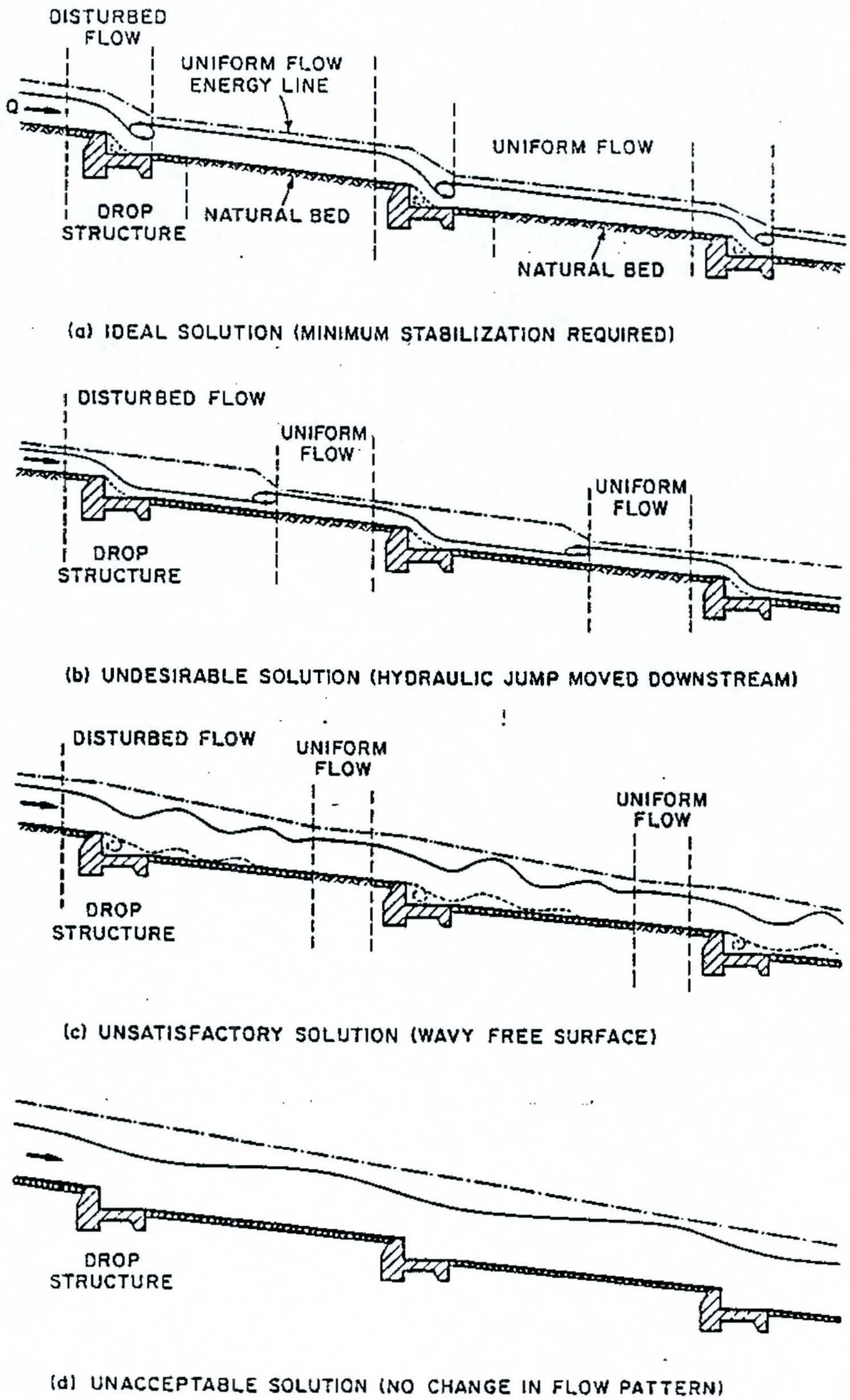


Figure 21. Different flow characteristics in cascade-type drop structures (after von Hausler 1976).

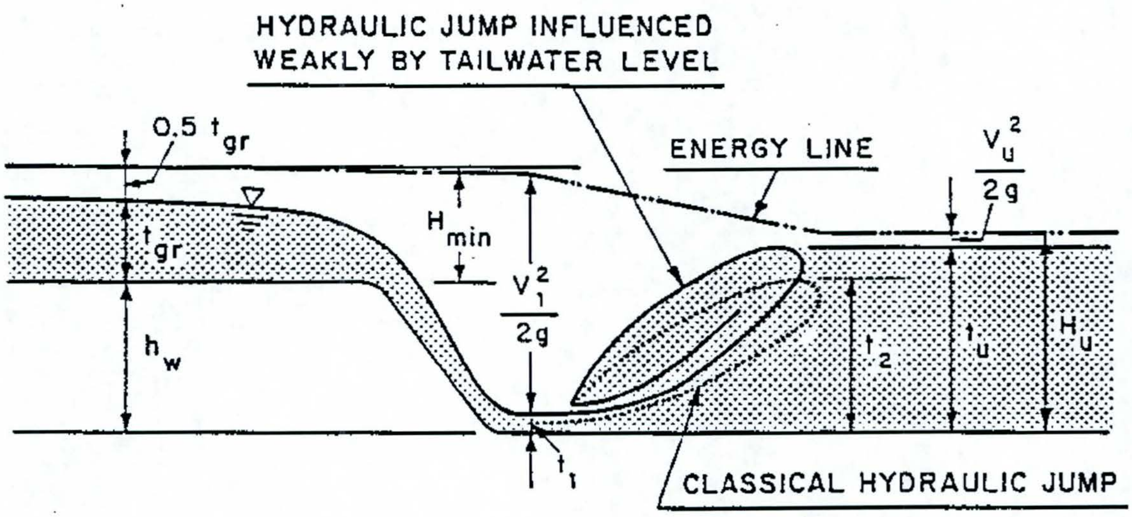


Figure 22. Definition sketch of hydraulic jump (after von Hausler 1976).

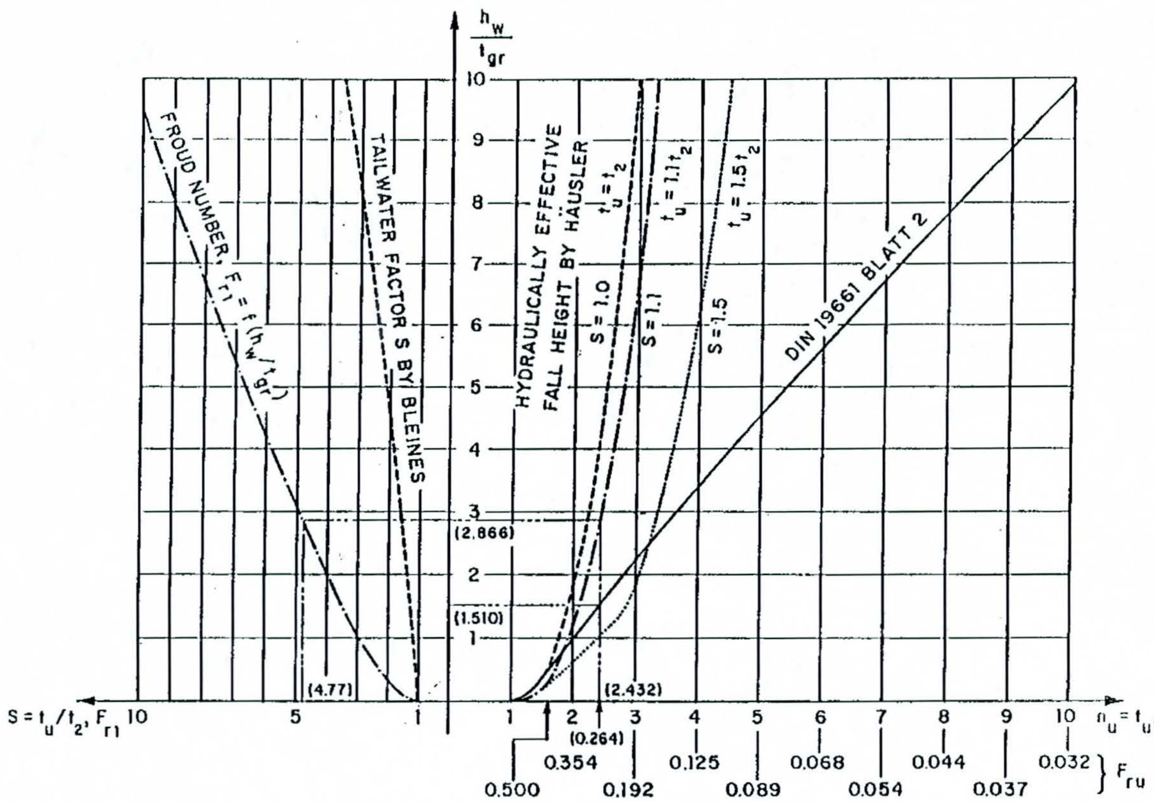


Figure 23. Design charts for drop structure (after von Hausler 1976).

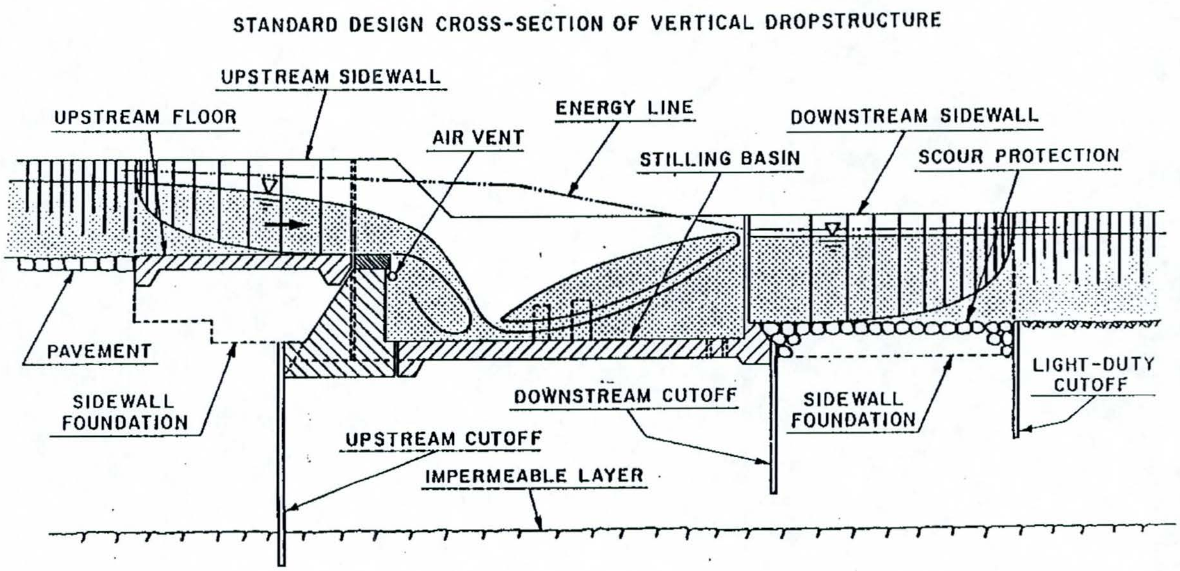


Figure 24. Standard design cross section of vertical drop structure (after von Hausler 1976).

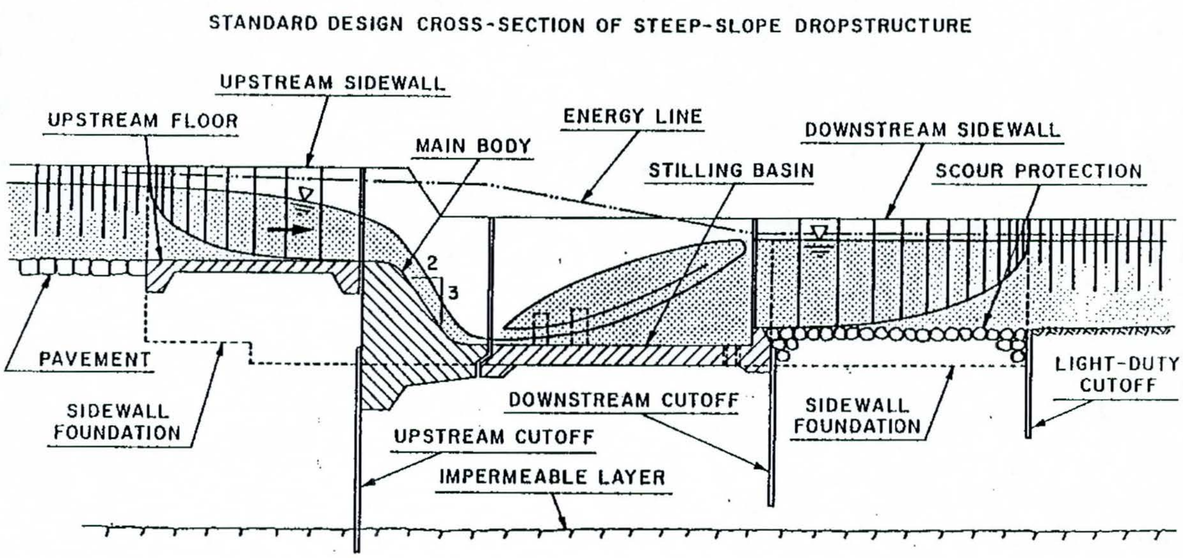


Figure 25. Standard design cross section of steep-slope drop structure (after von Hausler 1976).

D. GUIDELINES PROPOSED BY SMITH

Smith 1985 describes his design procedures for stone-based vertical drop structures based on his experimental data using three different sizes of quite uniformly distributed stone materials (i.e., 6.4 mm, 12.5 mm, and 27.1 mm). Figure 26 is a definition sketch for the variables. Figure 27 reproduces the design charts proposed by Smith and Strang 1967. In order to demonstrate the use of these charts, the following hypothetical case has been considered: P = 1.07 m; h_2 = 0.90 n; d_m = 0.18 m; and q = 1.0 m^2/s. Smith suggests that C = 1.84 for vertical weirs, and 1.70 for free overfalls in $H = (q/C)^{2/3}$ (SI units). In this case, H can be determined to be 0.67 m; hence, H/P = 0.63; P/d_m = 6.0; and h_2/P = 0.84. From Figure 27, d_s/P = 0.58; hence d_s = 0.62 m.

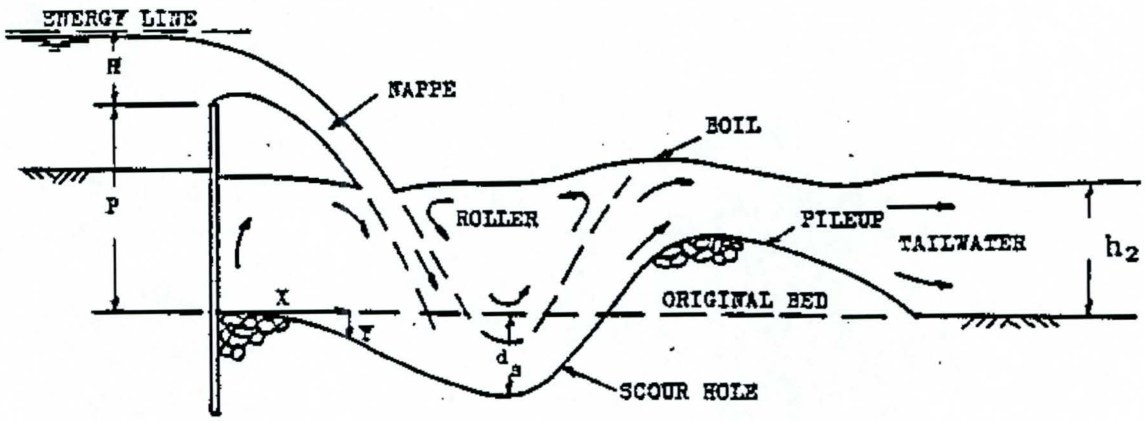


Figure 26. Definition sketch of stone-bed vertical drop structure (after Smith 1985).

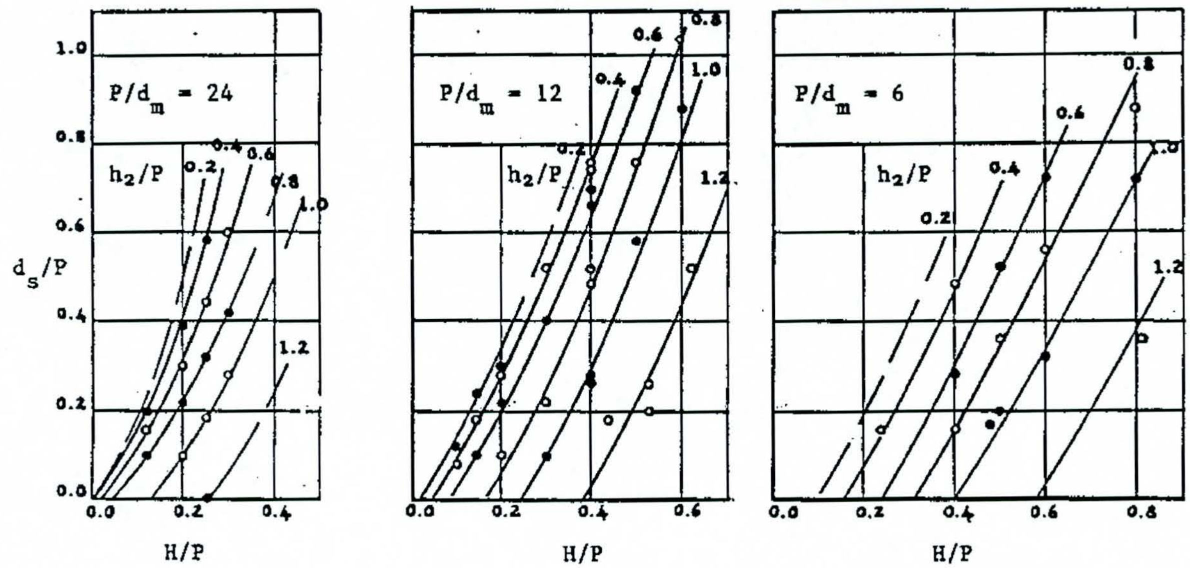


Figure 27. Charts for scour-depth prediction in stone-bed vertical drop structure (after Smith and Strang 1967).

Smith recommends the following design layout: (1) A stone layer 1.5 d_s deep, or in no case less than d_s + 2 d_m, should be placed in an excavation at the base of the weir, with the top of the layer (2/3) d_s below the normal downstream channel bed; and (2) In obtaining d_s from Figure 27, P and h_2 should be measured to the surface of the stone layer as originally placed. Note that his design charts are only applicable to stone with a specific gravity of 2.65; however, the charts can be used for a different value of the specific gravity. The indicated value of P/d_m on the chart must be reduced by multiplying it by $(S_s-1)/1.65$, in which S_s is the specific gravity. Smith recommends that a filter layer be placed at the boundary of the excavated line to avoid migration of foundation material under the stone bed. Figure 28 shows the overall design criteria. His design procedure has been modified by the Denver Urban Drainage District for vertical riprap drop structures Their modified version with a trickle channel is shown in Figure 18. The side slopes in the basin are limited to a maximum of 4H:IV. According to McLaughlin Water Engineers (McLaughlin Water Engineers 1986), vertical riprap drop structures in the real world appear to be more satisfactory than sloping rock drop structures in the Denver Urban Drainage District area, although there are fewer existing vertical drop structures than sloping drop structures McLaughlin Water Engineers recommends that a trickle channel notch be provided in the weir crest and appropriate transitions to the downstream trickle be designed in order to reduce local aggradation and degradation problems near the basin. The mound produced downstream of the basin (Figure 26) should be adjusted to an acceptable level so that it will not act as a second drop structure during lower discharges because no protection is provided downstream from the mound. Note that Farhoudi and Smith (Farhoudi and Smith 1985) conducted a model study to define scour-hole geometries downstream from hydraulic jump.

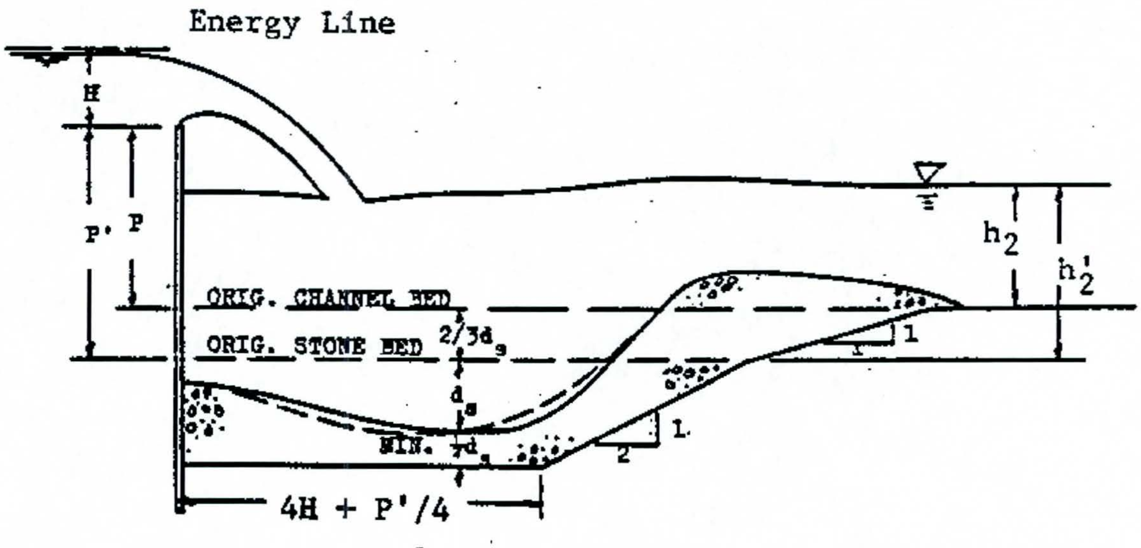
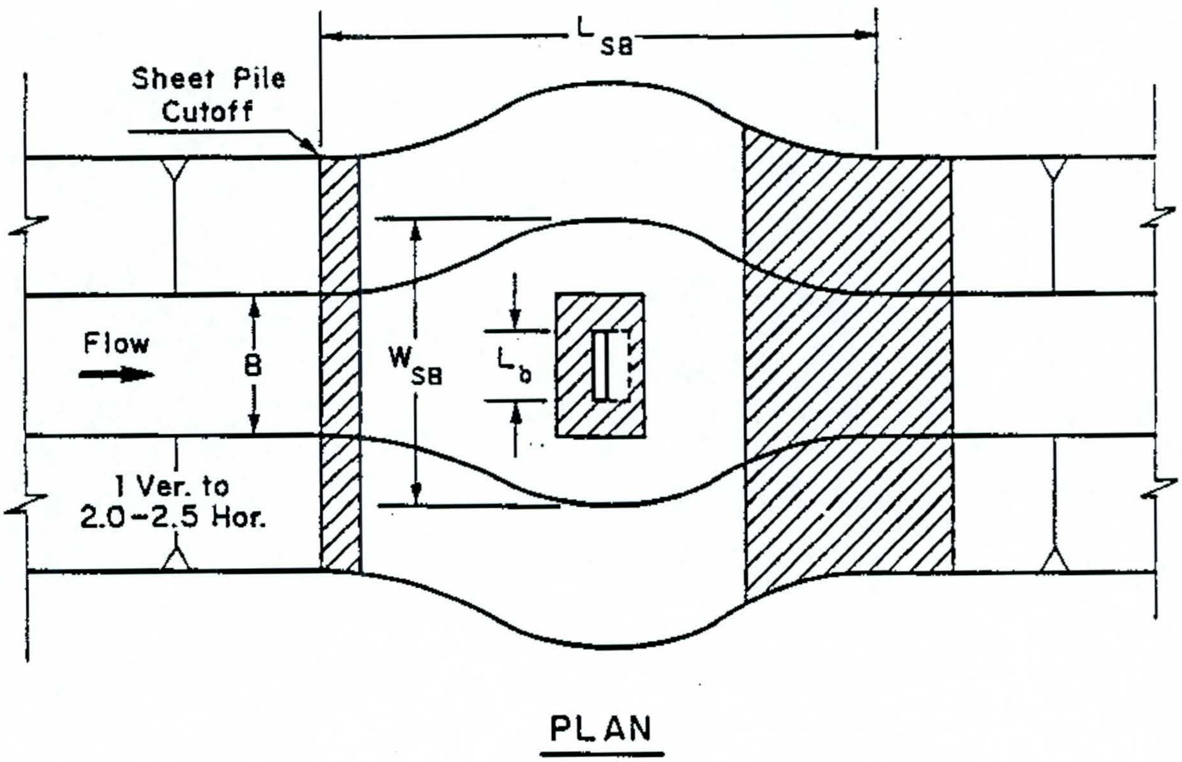


Figure 28. General design layout of stone-bed vertical drop structure stilling basin (after Smith 1985).

E. AGRICULTURAL RESEARCH SERVICE (ARS) GUIDELINES.

Little and Murphy (Little and Murphy 1982) conducted laboratory-scale model studies to develop low-drop, grade-control structures in which a relative drop height, H/Y_c, is less than or equal to 1.0, in which H is a bed-elevation difference between the upstream and downstream channels, and Y_c is a critical flow depth. Figure 29 shows tentative stilling-basin design guidelines developed by Little and Murphy. The stilling-basin width at the elevation of the weir crest, W_{SB}, was chosen to be $2B$, where B is the length of the weir crest. The stilling-basin length measured from the weir to the beginning of the downstream channel, L_{SB}, was chosen to be $2X_b$, where X_b is the distance from the weir to the crest of the first undulation of the hydraulic jump. Note that X_b was given as an empirical function of H/Y_c, and the stilling-basin depth, Y_{SB}, was set equal to Y_c+H. During the course of their model investigation, a highly organized stationary, undular wave pattern was found to persist for a great distance downstream, causing erosion problems downstream. To eliminate these undular waves, a new baffle concept was introduced, as can be seen in Figure 29. A baffle plate can be constructed by mounting horizontal steel plates or wooden planks onto H-piles driven into the channel. Their baffle-design guidelines are such that the baffle length, L_b, is $B/2$; the baffle height, H_b, is equal to Y_c; and the height of baffle above the weir crest, Y_b, is between $Y_c/4$ and $Y_c/3$.



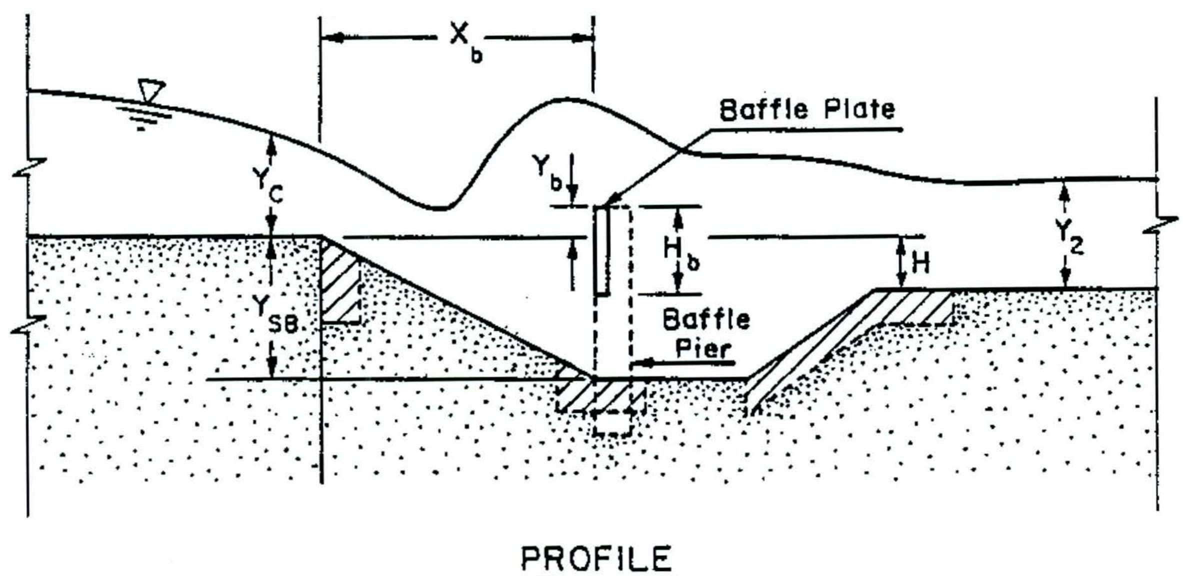


Figure 29. Definition sketch of ARS-type stilling basin and baffle plate (after Little and Murphy 1982).

Water Engineering & Technology, Inc. (Water Engineering & Technology, Inc. 1988) assessed the performance characteristics and the effectiveness of existing as-built, ARS-type, low-head drop structures in Mississippi, under field conditions. Nine low-drop, grade-control structures which were constructed between 1975 and 1985 were site-visited, and detailed hydraulic and sediment characteristics were analyzed using HEC-2 and HEC-6 programs. WET's field inspections revealed that the structures induced only limited sediment deposition upstream from them although the structures appeared to stabilize and maintain the channel grade. In all cases, scour holes several feet deep (measured relative to the sheet-pile weir crest), were found upstream of the low-drop structures due primarily to accelerated flow conditions there, which were also confirmed by the HEC-6 analysis. Their hydraulic analysis also indicated that the structures become submerged at relatively low, recurrence-interval flow events (say, less than 2 years against the design event of about 10 years), whereas the design criteria for ARS-type low-drop structures pertain to low tailwater conditions under which a weak hydraulic jump tends to form. It was found that the existing baffle pier or plate configuration blocks a considerable portion of the stilling-basin cross-section area, resulting in redistribution of flow directed toward both banks. This caused a downstream bank-stabilization problem. In two cases, an additional 100 ft of bank protection was needed downstream from the stilling basin, which made the construction costs extremely high. The structures investigated were found to have limited influence on enhancing channel-bank stability, although they prevent upstream migration of headcuts. WET concluded that the energy-dissipation features of the existing ARS-type low-drop structures are oversized. The design criteria for the existing ARS-type structures are applicable to conditions where critical flow conditions are created at the drop crest and the drop remains largely unsubmerged, while the existing structures are operating as submerged structures during the most major flow events. WET recommends re-evaluation of the existing criteria for the stilling-basin/baffle-pier configurations. The report by WET (Water Engineering & Technology, Inc. 1988) describes very comprehensive performance analyses of the nine existing ARS-type low-drop structures. It should be noted that Hite and Pickering (Hite and Pickering 1982) also conducted a laboratory model study of the proposed South Fork Tillatoba Creek drops in Mississippi to achieve optimized configurations of approach flow, stilling basin, and riprap protection for both low-stage and high-stage drop structures.

F. GUIDELINES FOR WEIR-TYPE DROPS PROPOSED BY ABLES AND BOYD

U.S. Army Corps of Engineers, Vicksburg District, pioneered in the use of low-water weirs in channel maintenance for the primary purpose of eliminating tree growth. Their experience in the late 1960's with low-water weirs (typically 5.5-ft high sheet-pile weirs) showed extensive scour holes immediately downstream from the weirs and subsequent deterioration of river banks downstream therefrom. Scour holes 25 to 30 ft deep were found to occur approximately 150 ft downstream from the structures (Ables and Boyd 1969). This was found to be caused by insufficient energy dissipation in the structure. In order to improve hydraulic performance of the low-water weirs and to obtain data on stable riprap designs for the stilling basin as well as the downstream channel, Ables and Boyd conducted laboratory model investigations using an undistorted geometric scale of 1:20. Figure 30 and Figure 31 show eight different channel-improvement plans. A straight anchor-pile weir is preferable to a segmented arch-pile weir, and a basin-type structure is preferable to a rock sloping-type structure. A basin length of 20 ft is adequate for a discharge ranging from 500 ft^3/s to 10,000 ft^3/s. The flared riprap section used in plan 6 was found to provide excellent slope protection downstream from the weir, but it would be expensive. Plan 8 is also listed to provide good slope protection at much less expense.

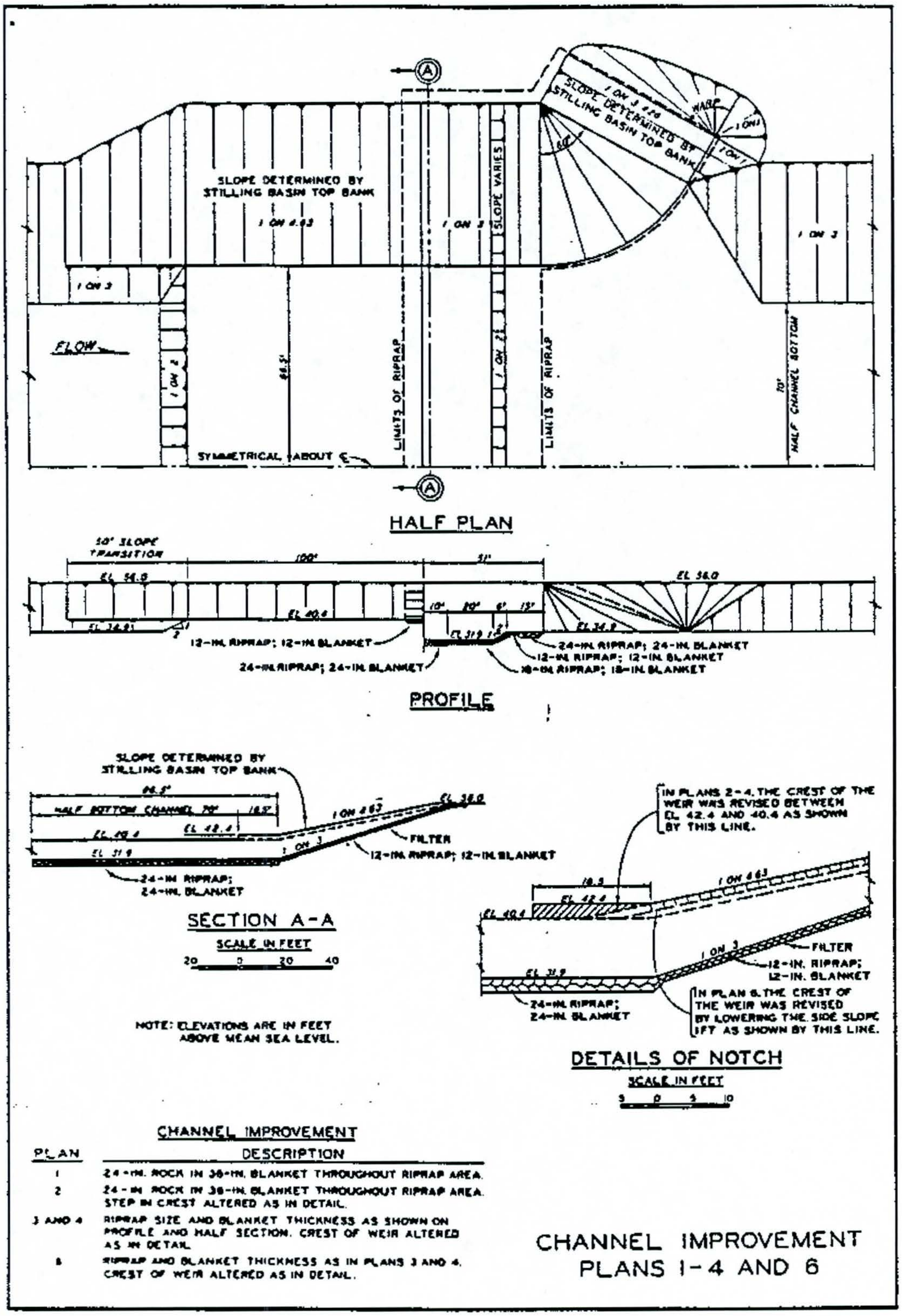


Figure 30. Channel-improvement plans using low-water weirs (after Ables and Boyd 1969).

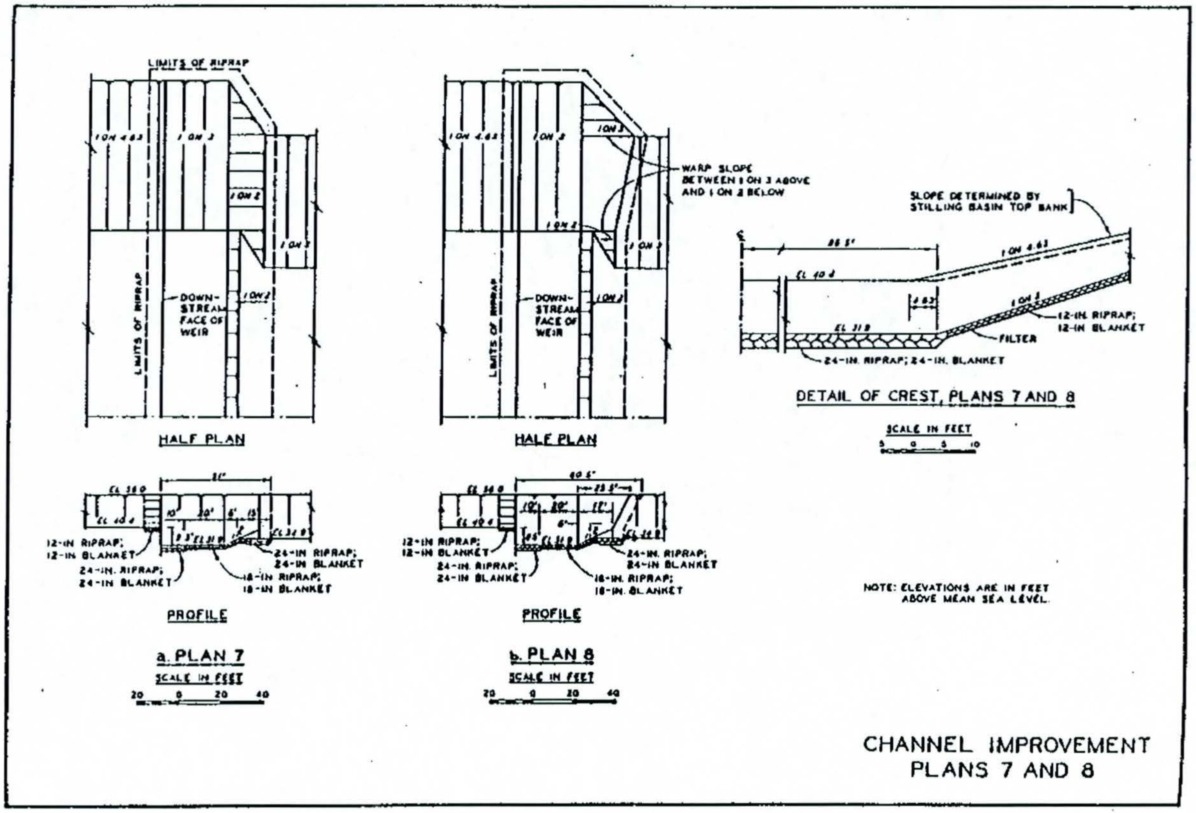


Figure 31. Channel-improvement plans using low-water weirs (after Ables and Boyd 1969).

Linder (Linder 1963) conducted a series of hydraulic model tests to develop a stream-bed stabilizer using sheet piling and rock sills for the Floyd River flood control project in Sioux City, Iowa. Figure 32, Figure 33, and Figure 34 show the design guidelines developed by the U.S. Corps of Engineers (US Army Corps of Engineers, 1991).

IV. CASCADED DROP STRUCTURES AND THEIR PROTOTYPE EXPERIENCE

Higashi (Higashi 1982) proposes cascaded; low-head, grade-control dams in contrast to so-called large-scale "SABO" dams (Japanese Ministry of Construction 1976, Japanese Ministry of Construction 1985). Tall SABO dams are generally constructed in narrow valley sections for the purpose of trapping upstream sediment and consequently, result in excess local-scour problems immediately downstream from them. Therefore, most SABO dams require smaller auxiliary dams downstream to control overscouring problems (Hayashi 1977; Japanese Ministry of Construction 1985). Higashi's idea is to consider this auxiliary dam as a main dam for sediment control and utilize a low-head, cascaded dam system as a grade-control system, instead of constructing a single, large-scale, SABO-type dam. On the basis of small-scale laboratory model tests and extensive field experience, Higashi derived general guidelines requiring that there be at least three low-head dams in a group; that they be spaced at about 50 meter intervals; and that their effective heights be 1-2 meters. One of his prototype experiences with the Nagashiri River in Japan is shown in Figure 35. The drainage area of this river is about 700 ha, and the original mean bed slope of the river section for which eleven cascaded low-head dams were constructed was about 3%. The logging road along the right bank used to be damaged frequently by floods when the bank caved in. As can be seen in Figure 35, each dam interval was chosen to be about 50 meters; the dam height varied from 1.5 to 4.0 meters; and the crown-to-crown slope (S_0) was designed to be somewhere between 2 to 3%. As shown in the section profile, which was measured one year after construction, the bed seemed to have stabilized rather quickly although several sections, such as sections between 4 and 5, and 9 and 10, still needed to be filled in. All these low-head, concrete dams were constructed in an L-shaped profile, some with bank retaining walls. Although Higashi's approach lacks basic hydraulic analyses, the successful field experience reported seems to highlight his creative, engineering achievement.

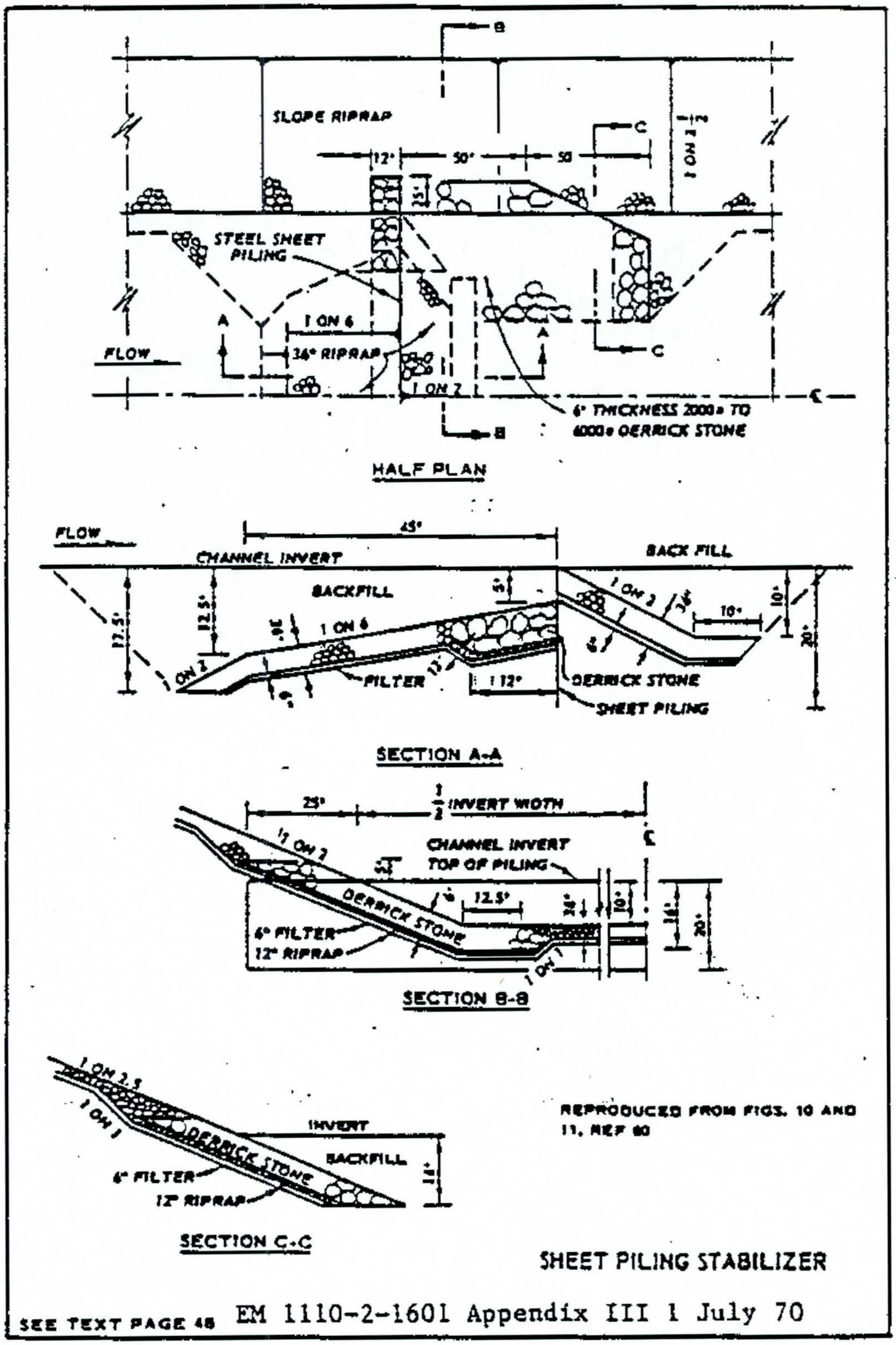


Plate 40

Figure 32. Corps of Engineers design layout of sheet-piling stabilizer (after US Army Corps of Engineers, 1991).

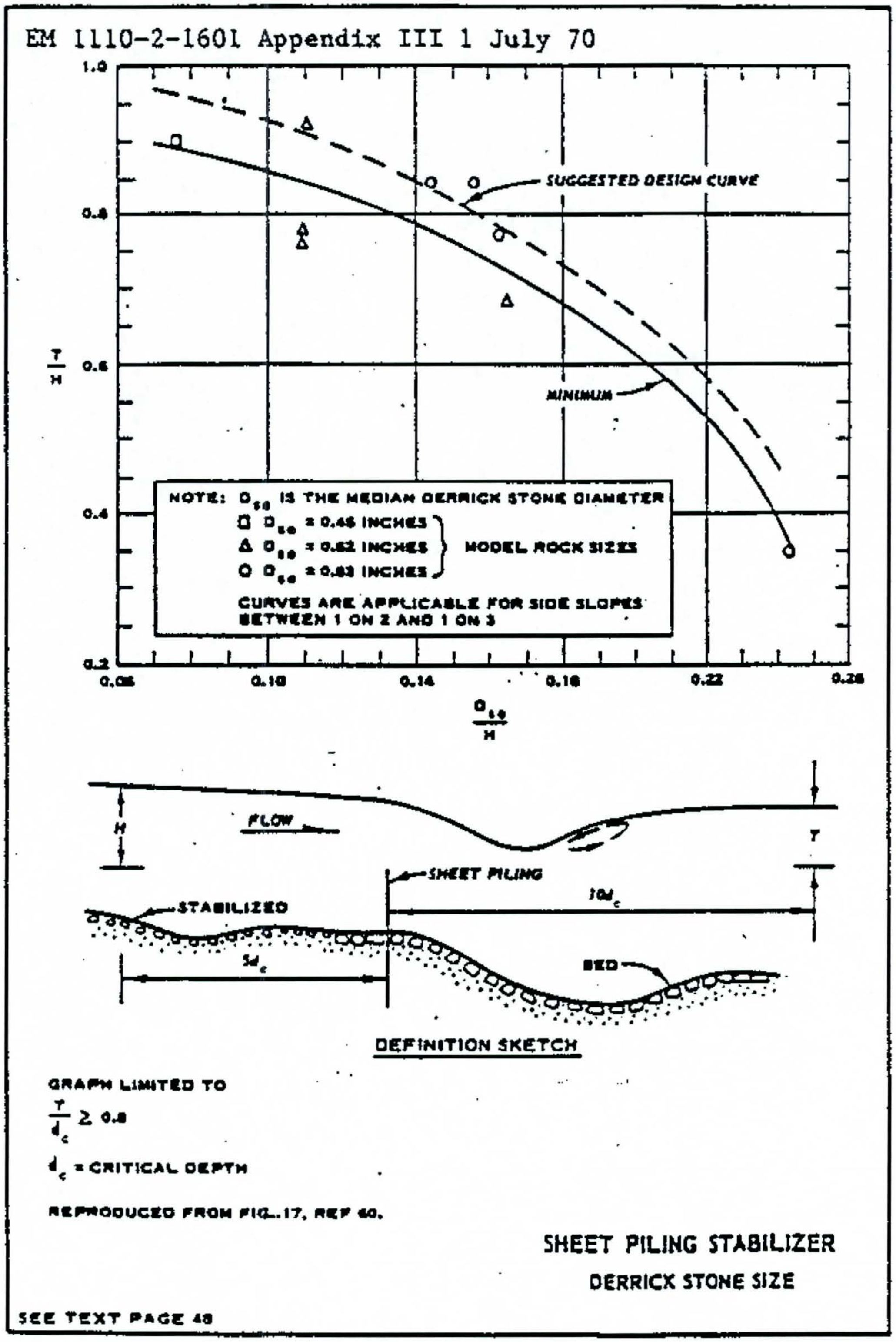


Plate 41

Figure 33. Corps of Engineers derrick-stone design parameters for sheet-piling stabilizer (after US Army Corps of Engineers, 1991).

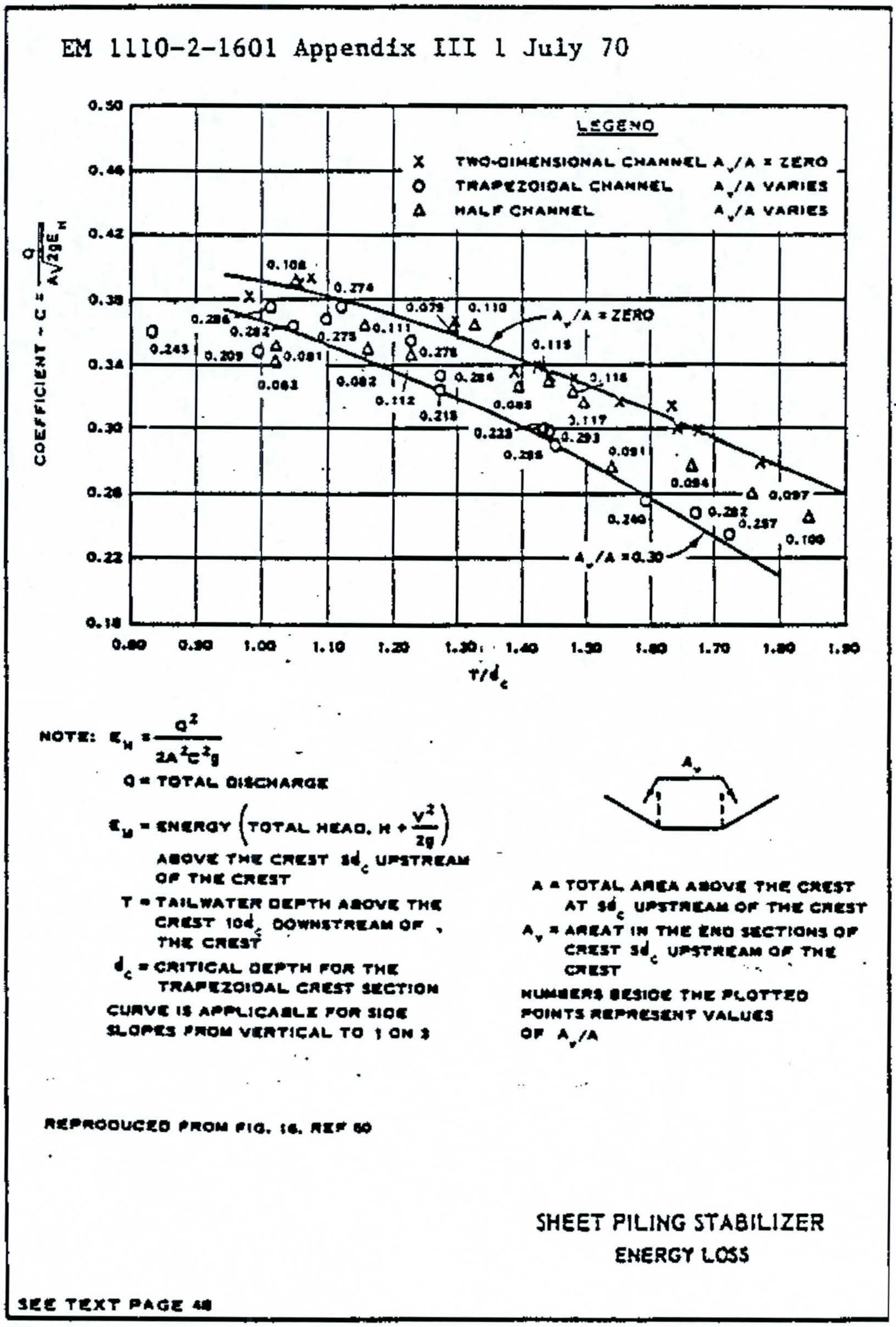
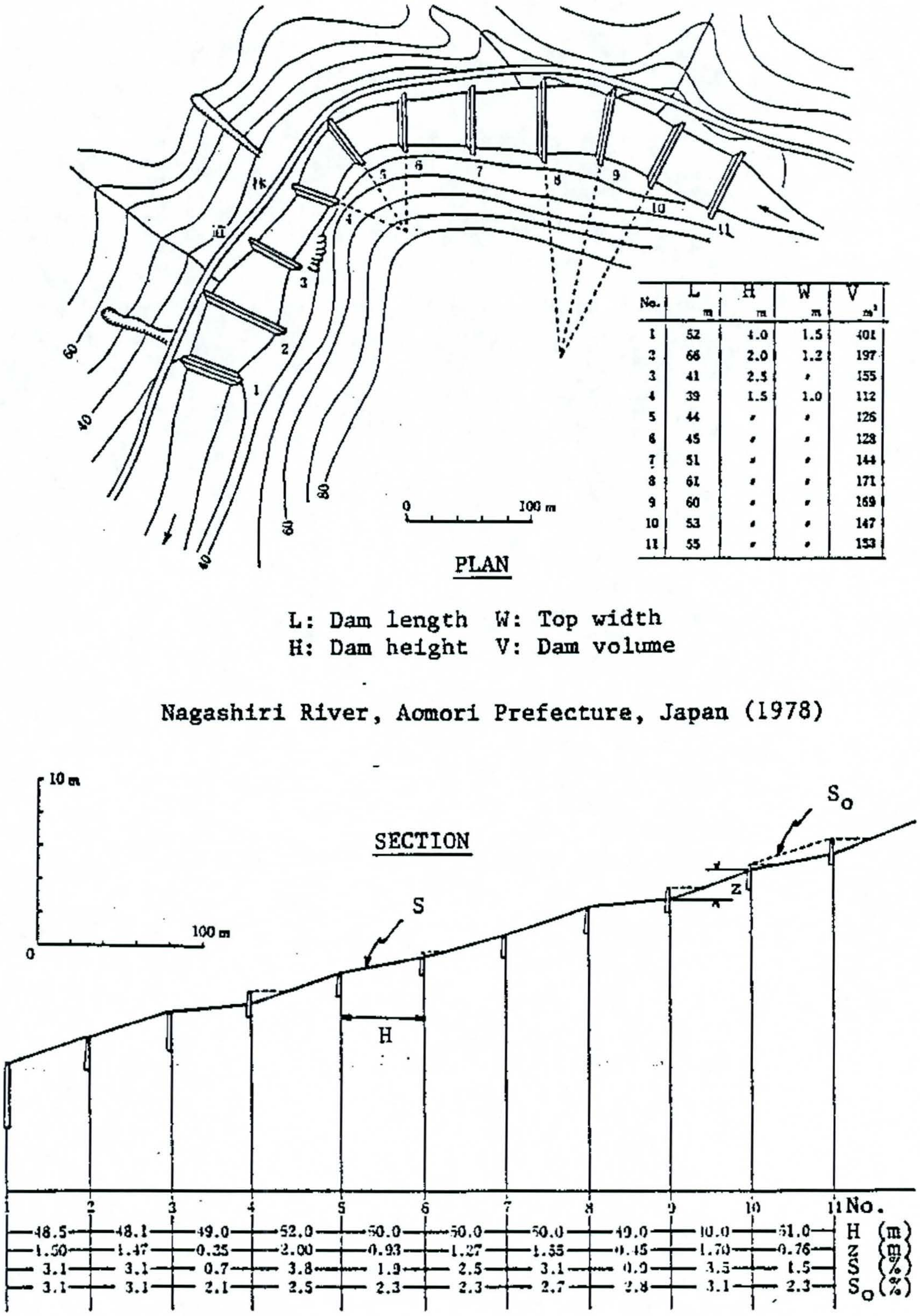


Plate 42

Figure 34. Corps of Engineers energy-loss design parameters for sheet-piling stabilizer (after US Army Corps of Engineers, 1991).



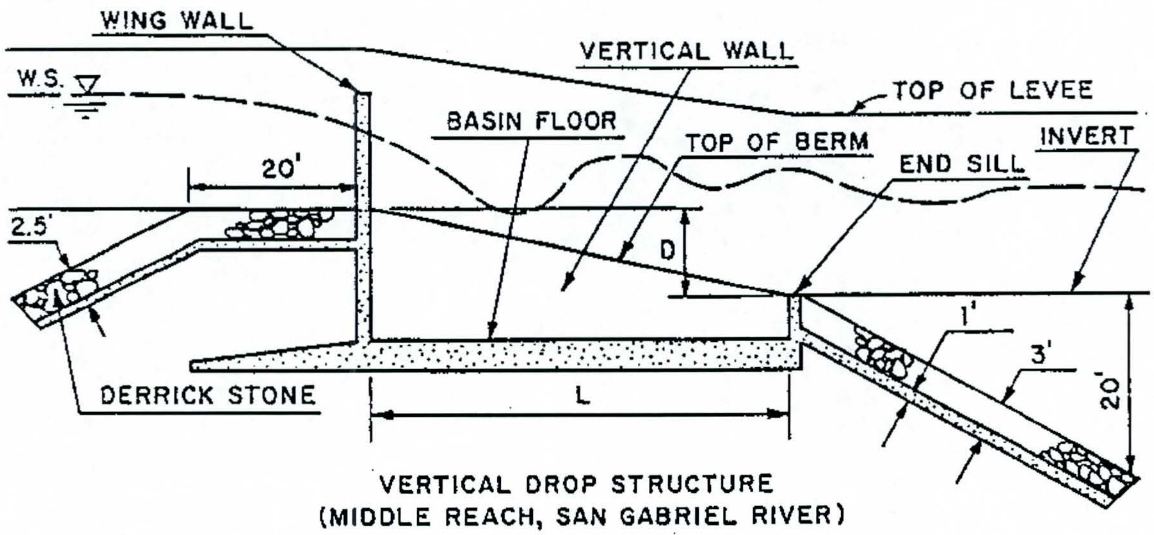
No.	L m	H m	W m	V m³
1	52	4.0	1.5	401
2	66	2.0	1.2	197
3	41	2.5	〃	155
4	39	1.5	1.0	112
5	44	〃	〃	126
6	45	〃	〃	128
7	51	〃	〃	144
8	61	〃	〃	171
9	60	〃	〃	169
10	53	〃	〃	147
11	55	〃	〃	153

Figure 35. Prototype layout of cascade-type low-head drop structures (after Higashi 1982).

In regard to cascaded drop structures, Yazawa (Yazawa et al. 1986) conducted laboratory model tests to study the effect of a series of submerged, lateral wing dikes in producing a stable low-flow channel between two drop structures Generally, river-bed stabilization structures are designed against large design floods, which results in formation of point bars within the channel during flows lower than the design discharges (Yazawa et al. 1986; and Kikkawa 1985). This means that meandering flow patterns tend to develop at intermediate discharges, resulting in flow concentration toward banks In many cases, expensive bank-protection works become necessary. Yazawa et al. 1986 focused on producing a compound channel between drop structures by means of wing dikes so that low- or intermediate-flows can pass through the straight, central portion of the channel without causing bank-erosion problems. Their preliminary results recommend that lateral dikes, a height equal to about one half of the uniform depth, be placed along both banks at an interval of about twice the river width. They also recommend that each dike length be determined in such a way that the backwater depth just upstream from each dike becomes somewhere between 1.25 to 1.50 times the uniform depth. They provide an empirical relationship to determine the dike length in terms of the dike height and the channel width.

Robles (Robles 1983), and Wong and Robles (Wong and Robles 1971) address the topic of drop structures for grade control of major flood-control channels in Southern California, including the Los Angeles and San Gabriel Rivers. Along the San Gabriel River, three types of drop structures were constructed during the period between the mid 1940's and the 1960's, including a sloping drop made of grouted stones, a vertical drop with stilling basin, and a vertical drop with stilling basin and baffle blocks. There are ten sloping drops in the upstream reach whose bed material is characterized by coarse sand, gravel and cobbles. The drop height for each grouted cobble-stone structure is 10 ft. The channel width in this reach is about 1,000 ft and the crest width of the control section is about 500 ft for a design discharge of 98,000 ft^3/s. The design unit-width discharge was 188 $ft^3/s/ft$, and a critical depth of 10.5 ft was assumed at the crest. It was predicted that the chute flow would develop a velocity of 40 ft/s before the hydraulic jump took place. The sequent depth was designed not to exceed 85% of the tail-water depth. Each drop was followed by a stabilizing structure made of grouted stones. The thickness of the upstream apron of the stabilizer was 5 ft and that of the downstream was 10 ft. Although no model study was conducted for these structures, they seemed to function very well. Because of coarse bed materials, the crest trickle-flow notch (20 ft wide and 6 ft deep) had to be lined with steel rails for erosion protection.

The 6-mi long middle reach of the San Gabriel River between Santa Fe Dam and Whittier dam is controlled by 15 vertical drop structures, each drop being 10 ft. The design flood varied between 40,000 ft^3/s and 60,000 ft^3/s, and the channel base width varied from 300 ft to 450 ft. The physical model study found that the initial design, based on the criteria proposed by Morris and Johnson (Morris and Johnson 1943) without baffle blocks, was inadequate to handle the design discharge. The final design developed from the model study included a rather extensive downstream derrick-stone apron with a slope of 2H:1V in order to cope with local scour due to roller actions. The 11-mi long lower reach was controlled by means of seven vertical drops with a base width ranging from 240 ft to 550 ft. The model study found that a row of baffle blocks, as shown in Figure 36, was needed. These vertical drop structures were designed based on the criteria developed by Blaisdell and Donnelly (Blaisdell and Donnelly 1951, Blaisdell and Donnelly 1966), and Donnelly and Blaisdell (Donnelly and Blaisdell 1954). Because of the baffle blocks, the downstream scour hole was much smaller than that in the case of the middle reach, resulting in a reduced amount of derrick stone in the downstream apron. According to Robles (Robles 1983), practical experience with the San Gabriel River drop structures showed very encouraging results with only minor localized scours in those located in the middle reach. Robles (1983) emphasizes the importance of the physical model study for satisfactory drop structure design. It should be pointed out that Woolhiser and Lenz (Woolhiser and Lenz 1965) applied a multiple-regression technique to obtain equilibrium, deposition bed slopes after construction of drops as a function of the initial bed slope, drop height, and channel width just above the drop structure. They found strong correlation's between these variables using field data of cascaded, debris check dams (drop height ranging from 7.6 ft to 18.0 ft) in the Los Angeles County Flood Control District as well as those of gully-control drop structures in Wisconsin (drop height up to 12.7 ft). Ferrell and Barr (Ferrell and Barr 1963) also presented their field experience with rather tall, debris-control, check darts in the Los Angeles County Flood Control District. They concluded that the most economic dam height is somewhere between 10 ft and 17 ft. Design guidelines for several types of check dams, including a concrete crib-type structure and a metal bin-type structure, are presented by them. Mizuyama and Bando (Mizuyama and Bando 1985) used a unique approach combining both a numerical model and a hydraulic-laboratory model to predict time-dependent, aggradation profiles above one particular SABO dam. Their technique was found to yield reliable design parameters economically and expeditiously.



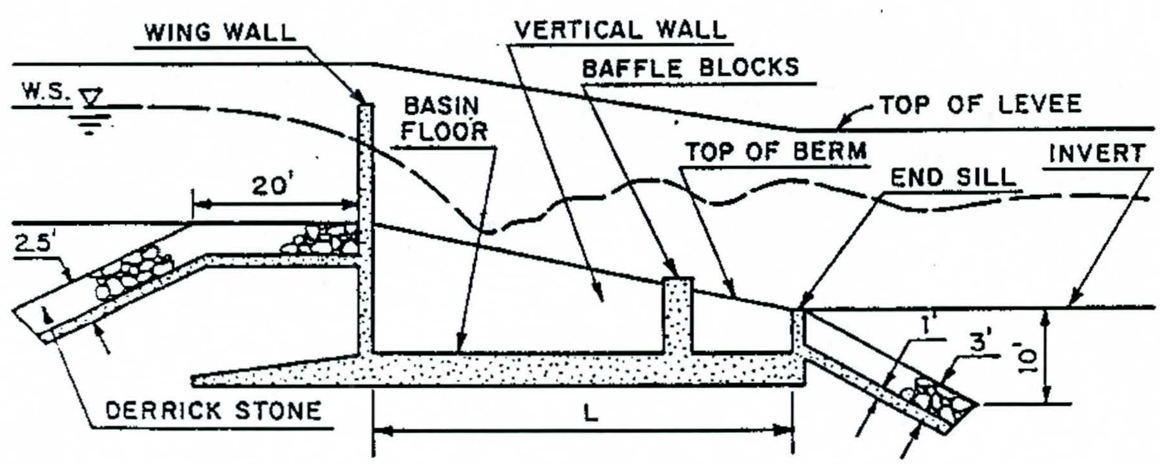


Figure 36. Design layouts of vertical drop structres: San Gabriel River (after Robles 1983).

Turner and Mulvihill (Turner and Mulvihill 1987) discuss general design modifications to existing Lower Santa Aria River drop structures on the basis of model studies. The existing vertical drop structures were found to be inadequate to convey the increased discharges resulting from recent changes in the drainage area and more accurate predictions of potential floods than predicted when the old structures were constructed. In order to accommodate larger discharges, a parabolic crest at the drop, one or more rows of baffle blocks, and a sloping end sill were added to each of the drop structures on the basis of model studies using 1:25-scale physical models. The maximum drop height tested was 12.75 ft and the minimum was 6.79 ft with the invert bed slope of approximately 0.0018. Submerged hydraulic jumps were found to occur if the Froude number was larger than 2.5. General drop structure design characteristics derived at are shown in Figure 37. They recommend that spaces between blocks be equal to the block height and the width be between 0.75 to 1.0 times the height. From optimization tests, a basin length of $3.0d_2$ (d_2 is defined in Figure 37) was found to produce minimum, acceptable scour depth downstream from the drop structure. Note that Tung and Mays (Tung and Hays 1982) proposed an optimization technique in designing stilling basins that minimizes construction costs and optimizes the hydraulic performance. The design chart for typical vertical drop structures recommended by the Office of the Chief of Engineers (US Army Corps of Engineers, 1991) is shown in Figure 38.

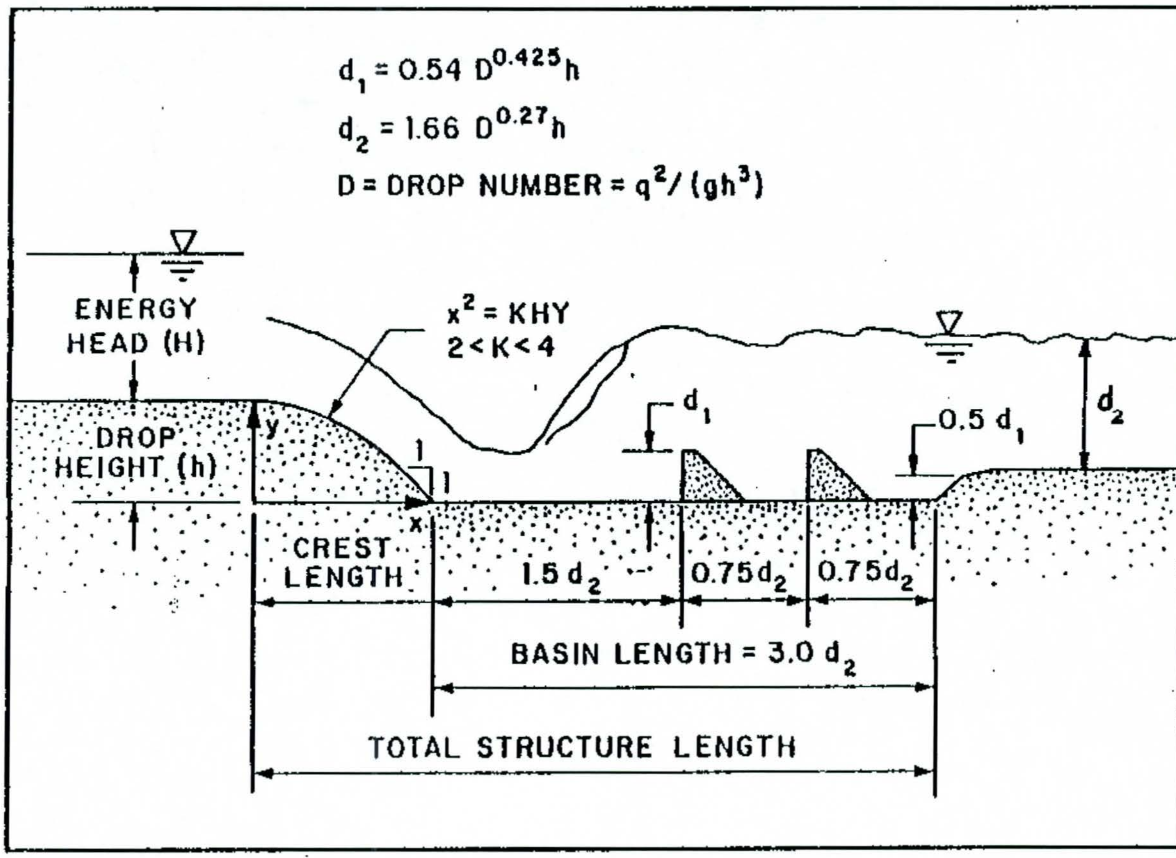


Figure 37. Design layout of drop structure (after Turner and Mulvihill 1987).

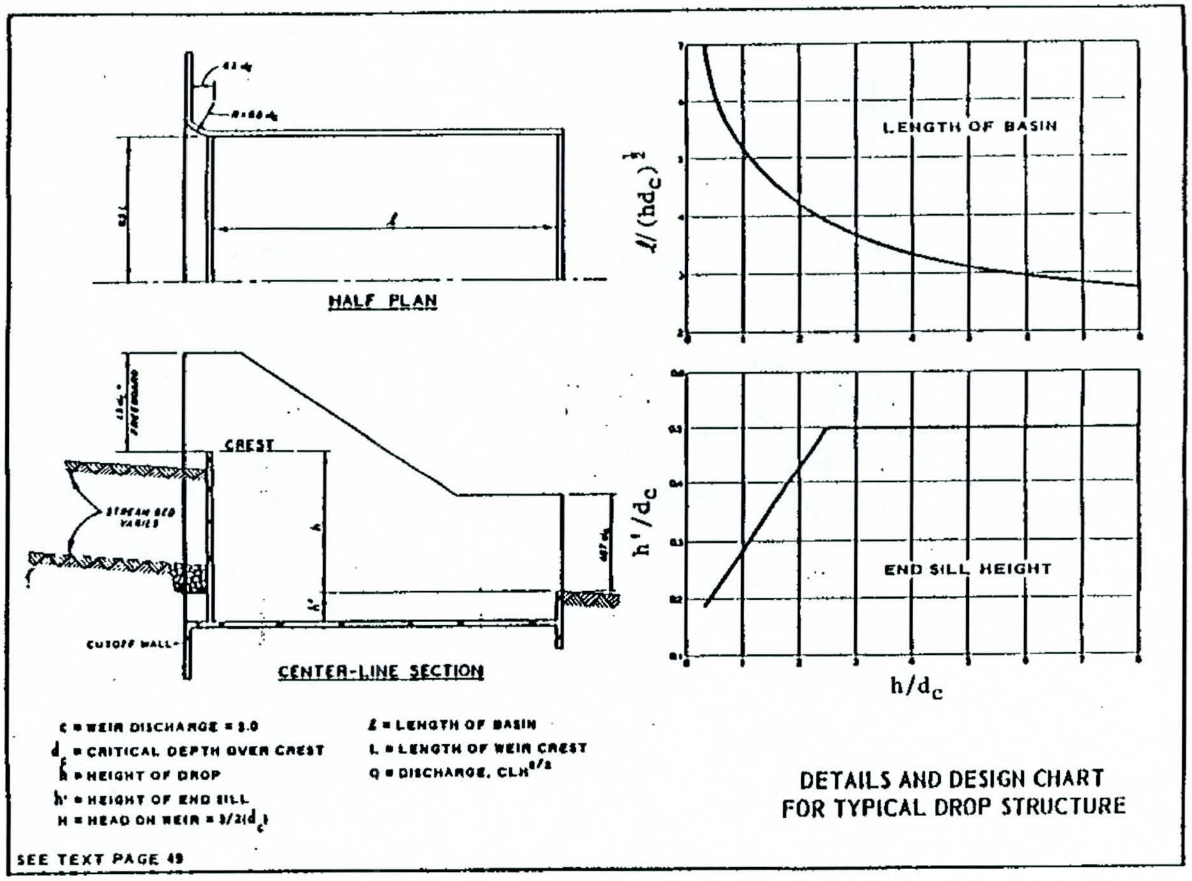


Figure 38. Corps of Engineers design charts for vertical drop structures (after US Army Corps of Engineers, 1991).

Whittaker and Jaggi (Whittaker and Jaggi 1986) present design criteria on the use of a series of sloping sills (Blockschwellen) to stabilize a long reach of steep alluvial channels. Their idea relies on producing a series of longitudinally stable river-bed segments by means of large rock materials which are placed strategically along the steep river section; thereby producing local degradation below sloping sills. The evolution of a stable river-bed channel is depicted in Figure 39. For a given bed-material composition, a statically stable bed slope is first estimated, and a series of block sills are placed along the river reach. The local scour downstream from each sill naturally develops a much milder bed slope than the original steep bed slope. This process is very similar to that seen often as bed degradation below a newly constructed dam. In this particular case, flexible block sills settle as scour holes develop below sills, resulting in a series of short, segmentwise stable channels, as shown in Figure 39-(c). This type of drop structure appears to be gaining in popularity in Europe, particularly in Switzerland, West Germany, and France (Office Federal de l'Economie des Eaux 1982) in order to maintain streams under natural conditions instead of using concrete structures. Whittaker and Jaggi (Whittaker and Jaggi 1986) present their experimental results on various types of sloping sills; however, combinations of hydraulic parameters, bed-material transport characteristics, stability of rock, scour depth, etc., must be investigated further to obtain design criteria which are applicable to prototype conditions .

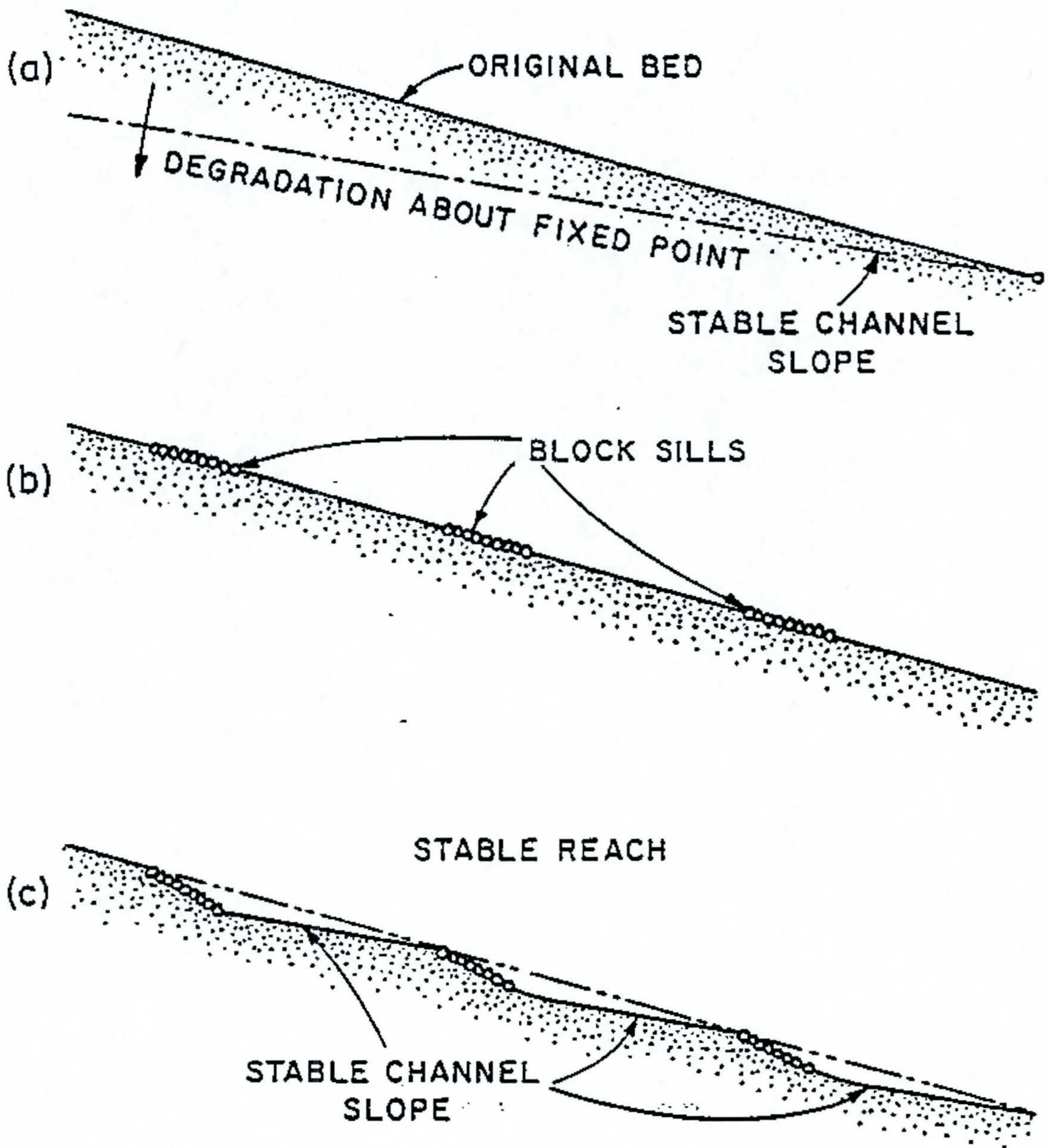


Figure 39. Design concept of cascade-type drop structures (after Whittaker and Jaggi 1986).

Richard (Richard et al. 1987) conducted a model study using a 1:40-scale model to obtain the highest level of recreational safety in a 10-ft tall low-head dam in Riverside Park in Grand Forks, North Dakota. On the basis of the model study, with a prototype discharge ranging from 1,000 to 6,000 ft^3/s, a three-level cascade-type drop structure was recommended. It was found that on each drop of the cascade, a submerged roller would form for a narrow range of flow, but this roller would not extend the full depth of the stream and should not create undercurrents which are unsafe to canoeists .

Krochin (Krochin 1961) also analyzed cascade-type drop structures using energy and rectangular-weir equations. He gives the following expression for the horizontal, jet-impingement distance, L_p. (see Figure 40)

$$L_p = 1.04q^{1/3}\left(y_1 + z_b + 0.22q^{2/3}\right)^{1/2} \tag{21}$$

L_p - Horizontal jet impingement distance (m)
q - Unit discharge (m3/s/m)
z_b - Drop height (m)
y_1 - Weir height (m)

In deriving equation 21, he assumed that the weir is sharp crested, and the brink depth, h_b, is $0.67H$. He also gives the horizontal stilling-basin distance, L_R, required for hydraulic jump to form:

$$L_R = 3.2d_2 \tag{22}$$

d_2 - Conjugate depth of d_1 (see Figure 40)

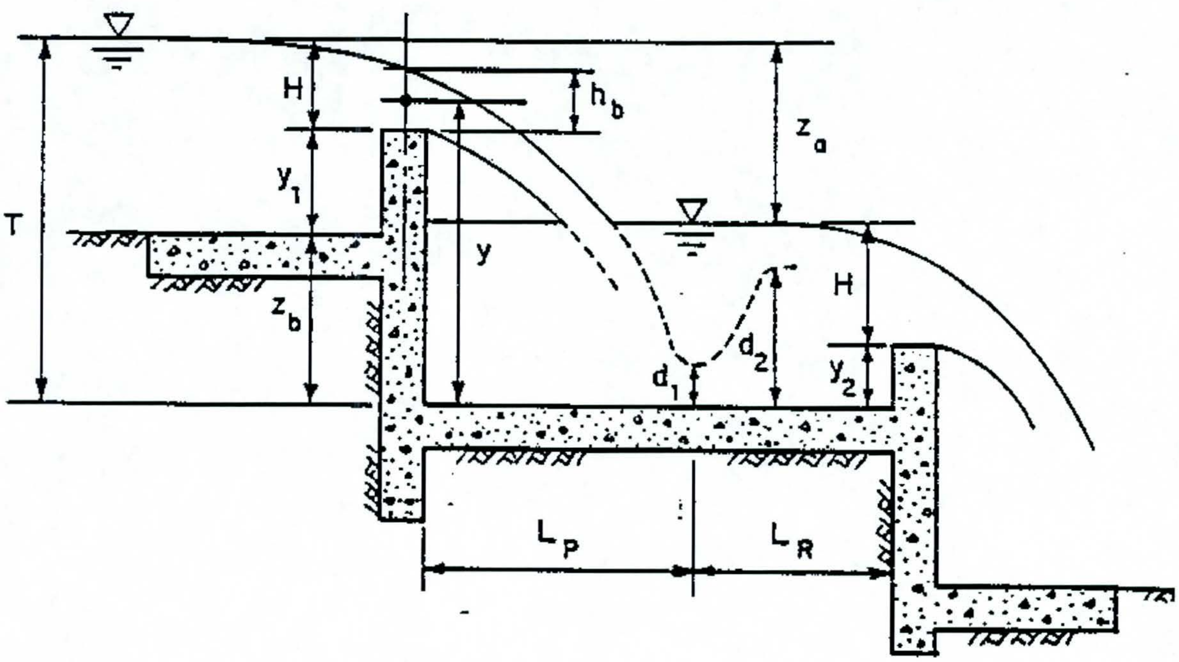


Figure 40. Definition sketch of cascade-type drop structures (after Krochin 1961).

V. MATHEMATICAL MODELS OF RIVER-BED AGGRADATION AND DEGRADATION

Recent years have seen important advances in the development of closed-form analytical techniques for prediction of alluvial stream-bed response to imposed changes in flow and sediment characteristics despite the fact that computer-based numerical simulation models for more complicated alluvial river systems have also been steadily under development (Holly et al. 1984). In this study, only analytical approaches which might be useful in investigating low-head, alluvial-channel, grade-control structures were reviewed.

Most of the analytical models developed so far may be classified into three distinguished groups, i.e., parabolic models, hyperbolic models, and kinematic-wave models. Each model specifies a mode of changes (aggradation or degradation) in longitudinal river-bed configuration. However, this presents a severe limitation to its application to grade-control structures because in real-world situations, aggradation occurs immediately upstream from a drop structure due to the sudden decrease in sediment-transport capacity of flow, while degradation takes place below the structure due to stoppage of the sediment supply to the downstream reach. Therefore, when a cascaded drop structure system is considered, both aggradation and degradation take place within each grade-control reach bounded by two drop structures. This implies that application of each analytical model to this particular case is practically impossible because the boundary conditions for both aggradation and degradation models cannot be explicitly specified. Nonetheless, some analytical models recently developed were reviewed.

All of the analytical models reviewed assume one-dimensional flow, weakly transient flow-resistance and sediment-transport predictors, and dependence of the local sediment discharge on the local hydraulic and bed-material conditions. Except for the kinematic-wave model (Matos-Silva 1986), most of the models combine flow-continuity, flow-momentum, sediment-continuity, flow-resistance, and sediment-transport equations, and obtain either parabolic-or hyperbolic-type partial differential equation. On the other hand, the kinematic-wave analysis assumes that the dependent variables, q_s and z, in which q_s is the sediment discharge per unit width and z is the bed elevation in the one-dimensional sediment-continuity equation, are interrelated (i.e., $q_s = q_s(z,x)$ in which x is the longitudinal distance), and their relationship is independent of the initial and boundary conditions (Matos-Silva 1986).

Adachi and Nakato (Adachi and Nakato 1969), de Vries (de Vries 1973), Ezaki (Ezaki 1983), Gill (Gill 1983a), Jain (Jain 1981), Jaramillo and Jain (Jaramillo and Jain 1984), Komamura (Komamura 1984), Lu and Shen (Lu and Shen 1986), Park and Jain (Park and Jain 1986), Philips and Sutherland (Phillips and Sutherland 1986), Soni, et al. (Soni et al. 1980) present their analyses of parabolic-type, one-dimensional, diffusion equations. All parabolic models assume a quasi-uniform flow condition with a uniform sediment size, and impose very restrictive assumptions for simplifying governing equations to obtain analytical solutions. All solutions are dependent on both imposed initial and boundary conditions.

Ribberink and Van Der Sande (Ribberink and Van der Sande 1985) analyzed aggradation problems due to sediment overloading (caused by either an increase in sediment discharge or a decrease in water discharge) using a hyperbolic model. Although they present analytical solutions for the linearized hyperbolic model, only approximate solutions were able to be obtained for either extremely snail or very large time scales. The hyperbolic model incorporates nonuniformity in flow distribution, and their analytical solutions are very complicated and not readily usable in their practical application. Hou and Kahawita (Hou and Kahawita 1988) also developed a nonlinear hyperbolic model for aggradation, and obtained the asymptotic equations to predict long-term solutions for a constant upstream overloading.

Matos-Silva (Matos-Silva 1986) proposes a new analytical solution, using the kinematic-wave approach, for a bed-degradation problem due to sudden curtailments of sediment discharge such as dam closure. His analytical solution was tested against numerical simulations conducted for the Missouri River for a range of water discharges and bed-sediment compositions, and reported to have yielded comparable results. The kinematic-wave approach was also applied to a bed-aggradation problem with limited verifications. Because of the simplicity of its solution, the kinematic-wave method seems to present a key advantage over numerical simulation models (Matos-Silva 1986). However, additional research is needed to verify its model with laboratory and field data. The kinematic-wave model appears to be best suited to investigate grade-control effects of cascaded-type drop structures. The degradation model can utilize some of the established scour-depth relationships reviewed in this study, and estimate its propagation rate downstream. An analysis of the aggradation aspect due to a damming effect of the low-head dam could probably be developed.

VI. CONCLUSIONS AND RECOMMENDATIONS

The principal conclusions and recommendations derived from the present, international literature survey on low-head, alluvial-channel, grade-control drop structures may be summarized as follows:

1. It appears that there are ample laboratory data and empirical relationships available to predict scour depth below either a vertical or a sloping drop structure. what is lacking is prototype verification of these relationships; in other words, a lack of effort in linking laboratory investigations with field engineering practices. A solid bridge to connect laboratory data and field experience must be established in order to obtain practically usable design criteria. In this context, the recent attempts by McLaughlin Water Engineers, Ltd. (1986) and Water Engineering & Technology, Inc. (1988) in critically assessing existing drop structures in the Denver Metropolitan Area and those in Mississippi, respectively, are the most welcomed signs of attempts at improving existing design guidelines.

2. There are no official standard guidelines for design of low-head drop structures. All documentation reviewed was more or less tentative or provisional, and furthermore, very site specific in nature. Only the Japanese Ministry of Construction (1976, 1985) seems to provide rather specific guidelines for vertical concrete-body drop structures. However, they are still provisional standards nonetheless. Based on their field evaluations, MWE (1986) and WET (1988) propose various site-specific design modifications. Their assessment experience should shared in the engineering society.

3. In many instances, satisfactory field performance was reported to be achieved by physical model investigations. In particular, Linder (1963), Ables and Boyd (1969), Hite and Pickering (1982), Little and Murphy (1982), Robles (1983), and Turner and Mulvihill (1987), among those which were reviewed, belong to this category.

4. There appears to be a strong need to investigate basic design criteria for stilling basins on proper energy dissipation under a wide range of tailwater conditions. The extent of bank stabilization downstream from a drop structure also appears to be an important area to be investigated. As reported by MWE (1986), incorporation of a trickle channel into a drop structure system seems to be another aspect of design concern.

5. Cascaded sloping sills (Blockschwellen in German) investigated by Whittaker and Jaggi (1986) appear to be extremely interesting because the evolution of a stable, longitudinal, river-bed profile could be achieved rather quickly under natural riverine processes in which flexible boulder sills settle naturally as scour holes develop below sills, yielding a much milder channel slope than the original one, as can be seen in Figure 39. There is potential for a successful analytical approach to attack this problem by predicting depth of equilibrium scour depth for a given bed-material size distribution. Given scour depth, the kinematic-wave analysis could be utilized to predict the evolution of the longitudinal bed profile.

SITING & SPACING

<This is the meat of the report. This section will put forth the latest methods (~5) for determining the spacing between structures and siting issues independent of spacing. Note that to many "Siting" includes "Spacing." We should settle on a definition for consistency. This section needs a discussion of Stable Channel Slope.>

<The following pages are from an Independent study Biedenharn did for Watson in 1994>

INTRODUCTION

One of the most challenging problems facing river engineers today is the stabilization of degrading channels. Channel degradation leads to the damage of bridges, culverts, and other structures with the general effect of disrupting transportation infrastructure. Utilities such as petrochemical transmission lines and power lines are also adversely affected by channel degradation. Channel degradation produces an over-heightened and over-steepened condition of the channel banks which often result in severe mass failures. The resulting channel widening and bank erosion cause severe land loss problems and endanger homes and other structures along top bank. The sediments that are eroded from degrading channels are deposited downstream and often impair flood control and navigation channels. Consequently, channel degradation is not simply a local problem which only affects a few landowners, but rather produces system-wide consequences that can affect all taxpayers.

Implementation of bank stabilization measures without proper consideration of the stability of the bed can result in costly maintenance problems and failure of structures. For this reason, it is essential to consider the stability of the bed as part of any bank stabilization scheme. Bank stabilization measures are generally appropriate solutions to local instability problems, such as erosion in bendways. However, when system-wide channel degradation exists, a more comprehensive treatment plan, which usually involves some form of grade control, must be implemented.

In the widest sense, the term "grade control" can be applied to any alteration in the watershed which provides stability to the streambed. By far the most common method of establishing grade control is the construction of in-channel grade control structures. Grade control can also be accomplished by altering the flow and sediment regime of the stream but these methods are less frequently used and will only be briefly discussed herein. There are many different types of hydraulic structures which serve the function of grade control. A partial list includes: bed stabilizers, data collection flumes, locks and dams, diversion structures, debris basins, low water weirs, culverts, check dams, and drop inlet structures. Many of these are multi-purpose structures in which grade control may only be a secondary by-product. This discussion will focus only on structures in which the primary purpose is to provide grade control for the stream.

This discussion is divided into four parts. The theoretical basis for providing grade control and an introduction to grade control concepts completes the first part. The second part classifies grade control structures into two basic categories and describes the performance of each type. The third part considers the primary factors for siting grade control structures. The last part presents various types of grade control structure designs.

GRADE CONTROL CONCEPTS

Channel degradation results when the sediment transport capacity in a reach exceeds the sediment supply. The actual channel processes are extremely complex but can, in most cases, be simplified and expressed reasonably well by Lane's qualitative relationship (Lane 1957) which states that:

$$QS \propto Q_s D_{50} \tag{23}$$

where Q is the discharge, S is the slope, Q_S is the bed material load, and D_{50} is the median size of the bed material. A stream is considered to be in equilibrium when these four controlling variables are balanced and the bed is neither aggrading or degrading. This equilibrium can be disrupted if one or some combination of the four variables are altered. Channel degradation occurs when the alteration results either directly or indirectly in a channel slope that is over-steepened ($QS^+ \propto Q_S D_{50}$) with respect to the other three variables, and must be reduced to re-establish equilibrium. This situation may be caused by a number of natural or man-induced factors such as a downstream channelization project, upstream dam construction, basin wide land use changes, or flow diversions.

Once degradation is initiated in a channel, it can be arrested by adjustments to one or some combination of the four Lane variables. This is often accomplished through the natural morphologic processes of the stream as it evolves toward a new equilibrium state. Unfortunately, this natural adjustment process is usually accompanied by unacceptable rates of erosion of the channel bed and banks, which may necessitate grade control being implemented. The following paragraphs describe how the four Lane variables can be altered by man to provide grade control in degrading streams.

One method to stabilize a degrading channel is to increase the amount of sediment supplied to the reach to a level commensurate with the over-steepened slope ($QS^+ \propto QS^+ D_{50}$), (ASCE 1975). while this is a theoretically possible alternative for providing grade control, it is seldom practical for three reasons: (1) it is extremely difficult to quantify with any reliability the precise amount of sediment that must be delivered; (2) even if the amount of sediment can be determined, there must exist an available source of material and a practical means of delivering it to the stream; and (3) the cost of such an undertaking is usually prohibitive, particularly in small flood control projects.

Bed stability can also be achieved by reducing the discharge through implementation of upstream flow control. This generally involves the construction of one or more dams in the upstream watershed. According to Lane's relationship, a reduction in discharge downstream of a dam can compensate for an over-steepened channel slope ($Q^- S^+ \propto Q_S D_{50}$). Therefore, the degradational potential in a channel due to excessive slope might be minimized or even eliminated by the reduced discharge. However, this scenario is based on the assumption that the sediment load (Q_S) remains unchanged. Unfortunately, this is seldom if ever the case since the trapping of sediment in the reservoir generally results in a reduced sediment supply downstream of the dam, thereby adding to the degradational tendency of the channel. Thus, the changes in the flow and sediment regime downstream of a dam tend to counteract each other. The ultimate channel response downstream of a dam will depend on the relative magnitude of the changes in the discharge and the sediment load and on the downstream watershed characteristics such as tributary inputs, geologic controls, bed and bank materials, and existing channel morphology. Consequently, the observed channel response downstream of a dam can vary considerably (Williams and Wolman 1984, Biedenharn 1983, Andrews 1986).

Because of the above mentioned complexities, there is very little definitive design criteria or guidance for the use of flow control as a means of providing bed stabilization. Consequently, it has not been used extensively. However, it has been used successfully in several watersheds in Mississippi by the Soil Conservation Service (SCS). Experience has shown that erosion of the bed and banks in these watersheds can be significantly reduced or eliminated when about 50 to 70 percent of the watershed area has been controlled by SCS dams (Water Engineering & Technology, Inc. 1989). Obviously these results can not be applied universally, but they do indicate that in some situations flow control can be a viable alternative for bed stabilization.

Although the above methods are theoretically possible and have been used in some instances, the most common method of providing grade control is to construct in-channel grade control structures. There are basically two types of grade control structures. One type of structure is designed to provide a hard point in the streambed that is capable of resisting the erosive forces of the degradational zone. This is somewhat analogous to locally increasing the size of the bed material (QS^{+} or $Q_{S}D_{50}^{+}$). For this discussion, this will be referred to as a "Bed Control Structure". The other type of structure is designed to function by reducing the energy slope along the degradational zone to the point that the stream is no longer capable of scouring the bed (QS^{-} or $Q_{S}D_{50}$). This will be referred to as a "Hydraulic Control Structure." The distinction between the processes by which these structures operate is important whenever grade control structures are considered. Because of the complex hydraulic behavior of grade control structures, it is difficult to develop an "ideal" classification scheme that will apply without exception to all situations. For many situations, the classification of a structure as either a bed control structure or hydraulic control structure is readily apparent. However there may be circumstances where a distinct classification of a structure as strictly a bed control or hydraulic control structure may be less evident and, in many cases, the structure may actually have characteristics of both. It also must be recognized that the hydraulic performance and therefore the classification of the structure, can vary with time and discharge. This can occur within a single hydrograph or over a period of years as a result of upstream or downstream channel changes.

PERFORMANCE OF GRADE CONTROL STRUCTURES

Although grade control structures are often considered very simple structures, the hydraulic processes associated with these structures are as complex and challenging as those of more elaborate hydraulic structures. The literature is replete with articles concerning grade control structures. However, most of these address the detailed hydraulic or structural design of the structure while much less is published on the actual mechanism by which these structures work. Consequently, there are many gaps in the knowledge concerning the behavior of these structures. While this discussion is not intended to fill in all the gaps, it will attempt to differentiate between the processes of two distinct types of structures and thereby explain in general terms the manner in which these structures function.

BED CONTROL STRUCTURES

In nature, channel degradation is often checked by outcrops of erosion resistant material in the streambed. These natural geologic controls serve as grade control by resisting the scouring forces of the degradational zone, thereby maintaining the bed elevation of the stream at that point. Therefore, these natural geologic controls may be viewed as Mother Nature's form of a bed control structure. Unfortunately, with time, these geologic controls may be eroded away, and then the degradational zone will continue its migration through the channel system.

The purpose of a bed control structure is to maintain the status quo of the upstream channel by forcing the degradational zone to occur at the structure where non-erodible materials will prevent scour of the bed. The weir crest of a bed control structure is generally constructed at or near the existing channel grade with a cross section that approximates the upstream channel dimensions. Consequently, these structures do not create any significant obstruction to flow which would require any additional energy to force the flow through the structure. Thus, a free overfall develops with an upstream M2 curve as the flow transitions from normal depth some distance upstream of the structure to critical depth in the vicinity of the weir. Figure 41 shows this configuration of weir and water surface profile. von Hausler (von Hausler 1976) refers to this type of structure without an upstream damming effect as "Absturz". With a bed control structure the flow lines upstream of the draw down zone are unchanged. Therefore, if the upstream channel is initially stable, then it will remain so, or if it is degradational, then it will continue to be degradational.

HYDRAULIC CONTROL STRUCTURES

As previously mentioned, a hydraulic control structure functions by reducing the upstream energy slope which renders the degradational zone inactive. Hydraulic control structures can also be used in situations where the bed has been previously degraded and it necessary to re-establish a desired grade in the channel. This is accomplished by building the bed up through sediment deposition upstream of the structure.

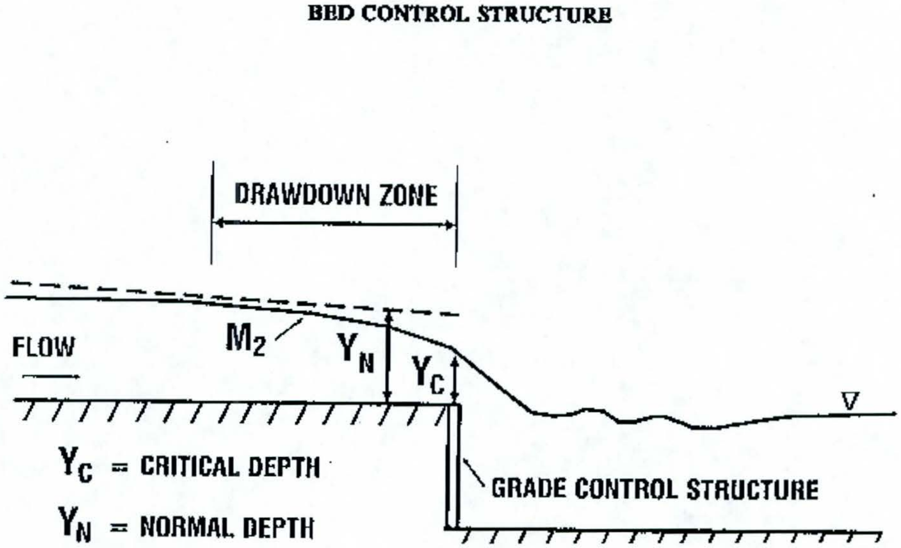


Figure 41. Bed control structure showing drawdown (M2 Curve) at weir.

While bed control structures are generally constructed upstream of the degradational zone, hydraulic control structures are situated downstream. Therefore, the hydraulic control structure must create a backwater situation with reduced velocities and scouring potential in order to be effective. By raising and/or constricting the weir section at the structure, a "choke" condition is attained which is analogous to the effect of a dam. This causes the upstream flow depth to increase as the stream attempts to attain sufficient energy to pass the flow through the structure. As a result, a hydraulic control structure is characterized by the presence of an upstream Ml curve. Figure 42 shows this configuration of grade control structure and water surface profile. Hausler (von Hausler 1976) refers to structures of this type with a dammed up drop as "Stutzschwelle." With the reduced velocities in the backwater area, the streambed is no longer subjected to the erosional potential of the degradational zone. At this point, the degradational zone is inactive and is referred to as having been "drowned out."

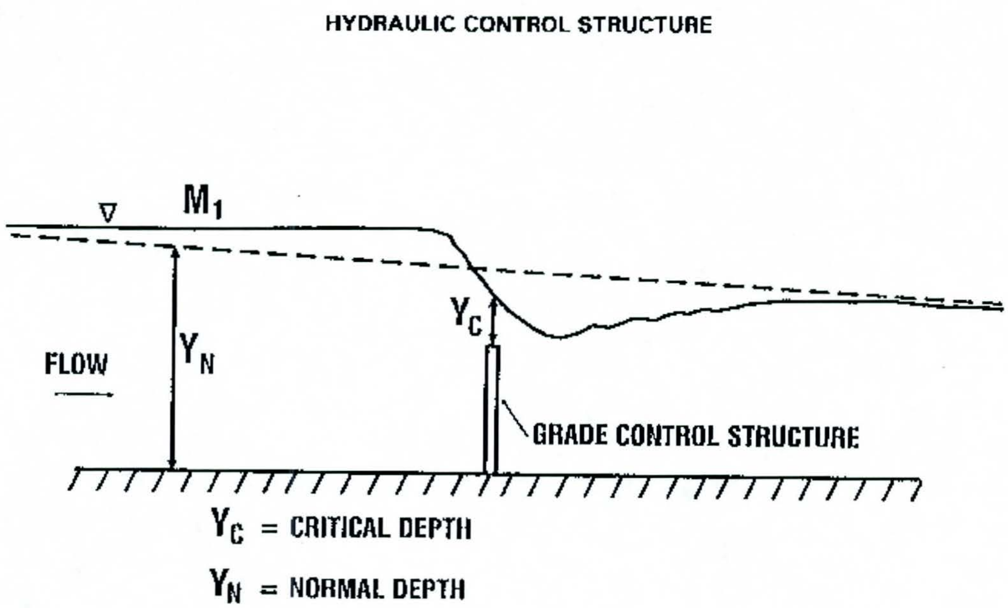


Figure 42. Hydraulic control structure showing M1 backwater curve.

The above discussion focused on the use of a hydraulic control structure to drown out a degradational zone. However, another aspect of hydraulic structures which must be considered is sediment deposition in the backwater zone. Sediment deposition upstream of a hydraulic control structure is the natural consequence of the reduced velocities due to the backwater effects. This deposition will continue until an equilibrium slope is developed upstream of the structure. Jansen (Jansen et al. 1979) describes the sedimentation response to the construction of a fixed weir, and the corresponding influence on the flow lines. Figure 43 illustrates the sedimentation response to a fixed weir. As will be discussed in the section HYDRAULIC CONSIDERATIONS, the determination of the ultimate equilibrium slope profile is extremely difficult and a major factor in the design of grade control structures.

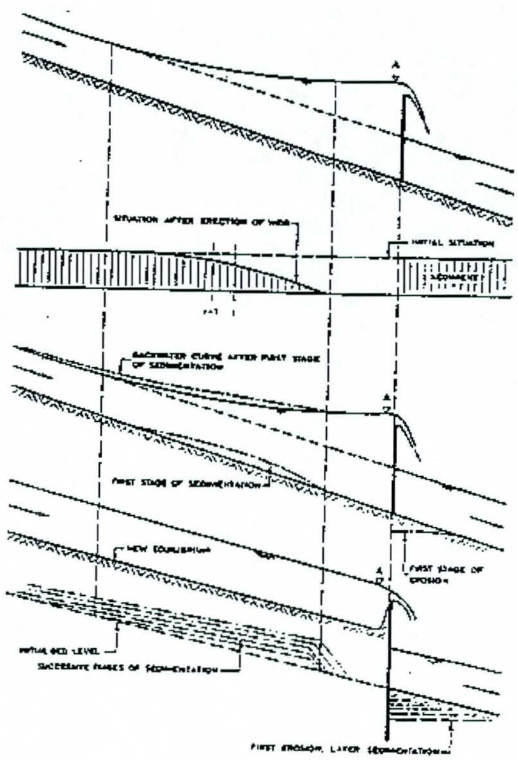


Figure 43. Successive stages of sedimentation following construction of a grade control structure (from Jansen et al. 1979).

DESIGN CONSIDERATIONS FOR SITING GRADE CONTROL STRUCTURES

Design considerations for siting grade control structures includes determination of the type, location along the stream, and the elevation and dimensions of one or more structures. Siting grade control structures is often considered a simple optimization of hydraulics and economics. However, these factors alone are usually not sufficient to define the optimum siting conditions for grade control structures. For example, hydraulic analysis may indicate that a stream reach requires three grade control structures, each with six feet of drop (see the section HYDRAULIC CONSIDERATIONS). However, six structures, each with three feet of drop, or various other combinations or drop height and number of structures may also be equally acceptable, hydraulically. In this case it would be a simple exercise to determine the least costly plan with respects to the hydraulic considerations; however, this plan would not address the various other issues that might exist in the project area. In practice the hydraulic considerations must be integrated with a host of other factors, which vary from site to site, to determine the final structure plan. Some of the more important factors to be considered when siting grade control structures are discussed in the following sections.

HYDRAULIC CONSIDERATIONS

One of the most important steps in the siting of a grade control structure or a series of structures is the determination of the anticipated drop at the structure. This requires some knowledge of the ultimate channel morphology, both upstream and downstream of the structure involves assessment of sediment transport and channel morphologic processes.

The hydraulic siting of grade control structures is a critical element of the design process, particularly when a series of structures is planned. The design of each structure is based on the anticipated tailwater or downstream bed elevation which, in turn, is a function of the next structure downstream. Heede and Mulich (Heede and Mulich 1973) suggested that the optimum spacing of structures is such that the upstream structure does not interfere with the deposition zone of the next downstream structure. Mussetter (Mussetter 1982) showed that the optimum spacing should be the length of the deposition above the structure which is a function of the deposition slope. Figure 44 illustrates the recommendations of Johnson and Minaker (Johnson and Minaker 1944). They recommend determining the most desirable spacing by extending a line from the top of the first structure at a slope equal to the maximum equilibrium slope of sediment upstream until it intersects the original stream bed.

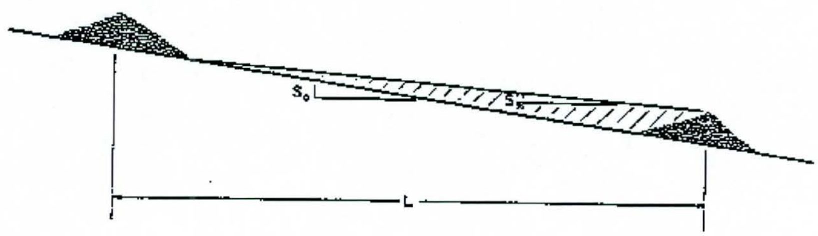


Figure 44. Spacing of grade control structure (from Mussetter 1982)

The theoretical hydraulic siting (spacing) of grade control structures is straightforward (Goitom and Zeller 1989).

$$H = (S_0 - S_f)x \tag{24}$$

H - Drop to be removed from reach
S_0 - Slope of the bed
S_f - Final or equilibrium slope
x - Length of the reach

The corresponding number of structures (N) required for a given reach is equally straightforward.

$$N = H/h \tag{25}$$

h - Structure height

Figure 45 and Figure 46 illustrate the hydraulic siting of a series of bed control structures using the above procedure. For this example, a degradational zone is located between points A and B, while the channel upstream and downstream of these points is in equilibrium (Figure 45). To determine the total amount of anticipated drop in the degradational zone, the equilibrium slope is projected upstream from point B. The resulting drop (H) at point A is 18 feet. Equation 25 reveals that three grade control structures, each with a design drop (h) of six feet, will be required to stabilize this reach.

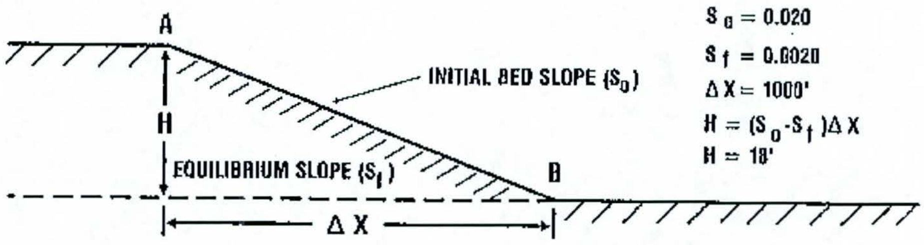


Figure 45. Initial condition of stream bed showing degradational zone between points A and B. Total anticipated drop in reach is calculated to be 18 feet.

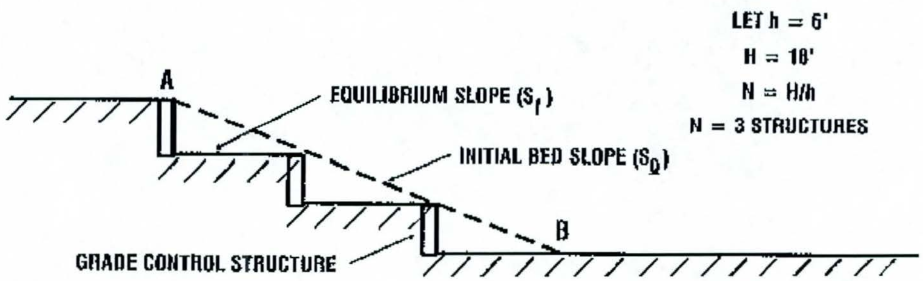


Figure 46. Stabilization of degradational zone using three bed control structures. Each structure has a design drop of six feet.

Figure 46 shows the layout of a series of bed control structures. The first structure is located at the point where six feet of degradation is anticipated based on the projection of the equilibrium slope upstream from point B. The weir crest of this structure is set at the initial bed elevation. Next, the equilibrium slope is projected upstream from the weir crest of this first structure and the process is repeated. Immediately following construction, these three structures will be little more than roughness elements in the streambed since there will be no drop across them. However, as the streambed continues to degrade, more and more drop will develop at each structure until the final equilibrium condition is obtained. It is important to note that while some bed degradation will occur between the structures, the stability (status quo) of the streambed upstream of point A is maintained.

Figure 46 shows bed control structures built at grade, and the bed degraded between each. In contrast, Figure 47 shows the hydraulic control structures constructed with a raised and possibly constricted weir crest that drowns out the degradational zone. In this instance, the first structure is located at point B with a weir crest six feet above the initial bed. The equilibrium slope is then projected upstream from the weir crest until it intersects the initial bed profile. The next structure is located at this point and the process repeated. Through time, the channel upstream of these structures should fill with sediment until the final equilibrium condition is reached.

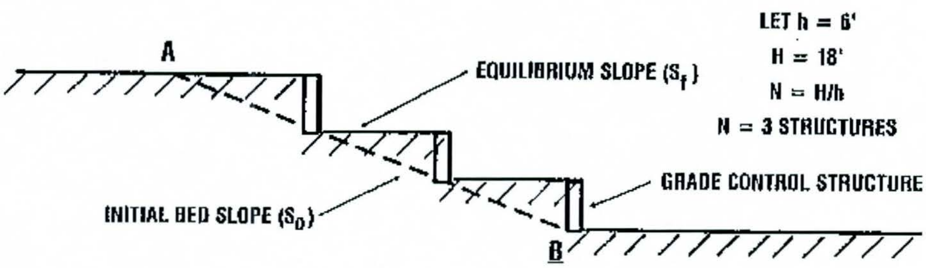


Figure 47. Stabilzation of degradational zone using three hydraulic control sstructures. Each structure has a design drop of six feet.

As indicated above, one of the most important factors when siting grade control structures is the determination of the equilibrium slope. Unfortunately, it is also one of the most difficult parameters to define with any reliability. Failure to properly define the equilibrium slope can lead to many problems. If the final channel slope is flatter than the predicted equilibrium slope, then the structures may be spaced too far apart. This can result in unacceptable degradation of the bed and inadequate design of the upstream structure which can lead to damage and possible failure of the structure. Conversely, if a steeper slope develops, then the structures may be spaced too close together. In this instance, the hydraulic performance of the upstream structure will be impaired by the sediment deposition.

The primary factors affecting the final equilibrium slope upstream of a structure include: the incoming sediment concentration and load, the channel characteristics (slope, width, depth, roughness, etc.), and the hydraulic effect of the structure. Another complicating factor is the amount of time it takes for the equilibrium slope to develop. In some instances, the equilibrium slope may develop over a period of a few hydrographs while in others, it may take many years. Another uncertain aspect in the determination of the final equilibrium profile is the ultimate headward extent. Many engineers maintain that the ultimate extent of sediment deposition will terminate at the upstream limits of the backwater effects of the structure, while others suggest that the sediment deposition will continue to build upstream through time.

This may be yet another example in river engineering where there is no single position that is applicable to all situations. A more detailed discussion of this phenomenon is found in Leopold et al. (Leopold et al. 1964).

There are many different methods for determining the equilibrium slope in a channel. These can range from detailed sediment transport modeling to less elaborate procedures involving empirical or process-based relationships such as regime analysis, tractive stress, or minimum permissible velocity. In some cases, the equilibrium slope may be based solely on field experience with similar channels in the area. The equilibrium profile upstream of grade control structures has been studied in considerable detail by numerous investigators: Harris 1901, Kaetz and Rich 1939, Johnson and Minaker 1944, Bermel and Sanks 1947, Amidon 1947, Heede 1960, Woolhiser and Lenz 1965, Sugio et al. 1973, and Mussetter 1982. Although the results of these studies varied considerably, it was generally found that the ultimate equilibrium slope (S_f) was less than the original slope (S_0) with S_f/S_0 generally falling within the range of 0.5 to 0.7. Regardless of the procedure used, the engineer must recognize the uses and limitations of that procedure before applying it to a specific situation. The decision to use one method or another depends upon several factors such as the level of study (reconnaissance or detail design), availability and reliability of data, project objectives, and time and cost constraints.

GEOTECHNICAL CONSIDERATIONS

The above discussion focused only on the hydraulic aspects of siting grade control structures. However, in some cases, the geotechnical stability of the reach may be an important or even the primary factor to consider when siting grade control structures. This is often the case where channel degradation has caused, or is anticipated to cause, severe bank instability due to exceedance of the critical bank Thorne and Osman 1988). When this occurs, bank instability may be common throughout the system rather than restricted to the concave banks in bendways. Traditional bank stabilization measures may not be feasible in situations where system-wide bank instabilities exist. In these instances, grade control may be the more appropriate solution.

Grade control structures can enhance the bank stability of a channel in several ways. Bed control structures indirectly affect the bank stability by stabilizing the bed, thereby preventing the banks from achieving an unstable height. With hydraulic control structures, two additional advantages with respect to bank stability are achieved: (1) bank heights are reduced due to sediment deposition, which increases the stability of the banks with regard to mass failures; and (2) by creating a backwater situation, velocities and scouring potential are reduced, which eliminates or minimizes the amount of basal cleanout of the failed bank material, thereby promoting self-healing of the banks. An example of the use of a hydraulic control structure to prevent head cutting and at the same time to promote bank stability is discussed by Biedenharn et al. 1990.

FLOOD CONTROL IMPACTS

Flood control and channel stability often appear to be mutually exclusive objectives. For this reason, it is important to ensure that any increased post-project flood potential is identified. This is particularly important when hydraulic control structures are considered. In these instances the potential for causing overbank flooding may be the limiting factor with respect to the height and amount of constriction at the structure. Grade control structures are often designed to be hydraulically submerged at flows less than bankfull so that the frequency of overbank flooding is not affected. However, if the structure exerts control through a wider range of flows including overbank, then the frequency and duration of overbank flows may be impacted. When this occurs, the impacts must be quantified and appropriate provisions such as acquiring flowage easements or modifying structure plans should be implemented.

Another aspect of flooding that must be considered when siting grade control structures is the safe return of overbank flows back into the channel. This is particularly a problem when the flows are out of bank upstream of the structure but still within bank downstream. The resulting head differential can cause damage to the structure as well as severe erosion of the channel banks depending upon where the flow re-enters the channel. Some means of controlling the overbank return flows must be incorporated into the structure design. One method is simply to design the structure to be submerged below the top bank elevation, thereby reducing the potential for a head differential to develop across the structure during overbank flows. If the structures exerts hydraulic control throughout a wider range of flows including overbank, then a more direct means of controlling the overbank return flows must be provided. One method is to ensure that all flows pass only through the structure. This may be accomplished by building an earthen dike or berm extending from the structure to the valley walls which prevents any overbank flows from passing around the structure (Forsythe 1985). Another means of controlling overbank flows is to provide an auxiliary high flow structure which will pass the overbank flows to a specified downstream location where the flows can re-enter the channel without causing significant damage (Hite and Pickering 1982).

ENVIRONMENTAL CONSIDERATIONS

The key phrase in water resources management today is "sustainable development" which simply means that projects must work in harmony with the natural system to meet the needs of the present without compromising the ability of future generations to meet their needs. Engineers and geomorphologists are responding to this challenge by trying to develop new and innovative methods for incorporating environmental features into channel projects. One method which is gaining widespread attention is the application of grade control structures in a much broader sense to provide for environmental sustainability as well as erosion control.

Grade control structures can produce positive environmental impacts on a channel system in a number of ways. By definition, grade control structures are normally placed in severely unstable stream reaches. By preventing the headward migration of zones of degradation, grade control structures provide vertical stability to the stream and reduce the amount of sediment eroded from the stream bed and banks. This not only protects the upstream reaches from the destabilizing effects of bed lowering, but also can minimize the sedimentation problems in the downstream reaches due to the reduced sediment loading. Therefore, the impacts of grade control structures are not restricted to a local area around the structure, but, rather, can have far reaching impacts on the whole channel system.

Grade control structures can also provide direct environmental benefits to a stream. Cooper and Knight (Cooper and Knight 1987) conducted a study of fisheries resources below natural scour holes and man-made pools below grade control structures in north Mississippi. They concluded that although there was greater species diversity in the natural pools, there was increased growth of game fish and a larger percentage of harvestable-size fish in the man-made pools. They also observed that the man-made pools provided greater stability of reproductive habitat. Shields et al. 1990 reported that the physical aquatic habitat diversity was higher in stabilized reaches of Twentymile Creek, Mississippi than in reaches without grade control structures. They attributed the higher diversity values to the scour holes and low-flow channels created by the grade control structures. The use of grade control structures as environmental features is not limited to the low-gradient sand bed streams of the southeastern US. Jackson (Jackson 1974) documented the use of Gabion grade control structures to stabilize a high-gradient trout stream in New York. She observed that following construction of a series of bed sills, there was a significant increase in the density of trout. The increase in trout density was attributed to the accumulation of gravel between the sills which improved the spawning habitat for various species of trout.

Adverse environmental impacts can also be associated with grade control structures. During the construction of any structure, there is always the potential for the destruction of riparian habitat that must be contended with. However, with grade control structures, these impacts are usually limited to a localized area at the structure as opposed to other types of channel improvement features (levees, bank stabilization, or channelization) where the habitat destruction may occur continuously over long reaches of stream.

Perhaps the most serious environmental impact of grade control structures is the obstruction to fish passage. In some cases, particularly when drop heights are small, fish are able to migrate upstream past a structure during high flows (Cooper and Knight 1987). However, in situations where structures are impassable, and where the migration of fish is an important concern, openings, fish ladders, or other passageways must be incorporated into the design of the structure to address the fish movement problems (Nunnally and Shields 1985). The various methods of accomplishing fish movement through structures are not discussed here. The reader is referred to Nunnally and Shields (Nunnally and Shields 1985), Clay (Clay 1961), and Smith (Smith 1985) for a more detailed discussion.

The environmental aspects of the project must be an integral component of the design process when siting grade control structures. A detailed study of all environmental features in the project area should be conducted early in the design process. This will allow these factors to be incorporated into the initial plan rather than having to make costly and often less environmentally effective last minutes modifications to the final design. Unfortunately, there is very little published guidance concerning the incorporation of environmental features into the design of grade control structures. One source of useful information can be found in the following technical reports published by the Environmental Laboratory of the Corps of Engineers, Waterways Experiment Station: (Shields and Palermo 1982; Henderson and Shields 1984; and Nunnally and Shields 1985).

EXISTING STRUCTURES

Bed degradation can cause significant damage to bridges, culverts, pipelines, utility lines, and other structures along the channel perimeter. Grade control structures can prevent this degradation and thereby provide protection to these structures. For this reason, it is important to locate all potentially impacted structures when siting grade control structures. The final siting should be modified, as needed, within project restraints, to ensure protection of existing structures.

It must also be recognized that grade control structures can have adverse as well as beneficial effects on existing structures. This is a concern upstream of hydraulic control structures due to the potential for increased stages and sediment deposition. In these instances, the possibility of submerging upstream structures such as water intakes or drainage structures may become a deciding factor in the siting of grade control structures.

Whenever possible, the engineer should take advantage of any existing structures which may already be providing some measure of grade control. This usually involves culverts or other structures that provide a non-erodible surface across the streambed. Unfortunately, these structures are usually not initially designed to accommodate any significant bed lowering and, therefore, can not be relied on to provide long term grade control. However, it may be possible to modify these structures to protect against the anticipated degradation. These modifications may be accomplished by simply adding some additional riprap with launching capability at the downstream end of the structure. In other situations, more elaborate modifications such as providing a sheet pile cutoff wall or energy dissipation devices may be required. Damage to and failure of bridges is the natural consequence of channel degradation. Consequently, it is not uncommon in a channel stabilization project to have several bridges that are in need of repair or replacement. In these situations it is often advantageous to integrate the grade control structure into the planned improvements at the bridge. If the bridge is not in immediate danger of failing and only needs some additional erosion protection, the grade control structure can be built at or immediately downstream of the bridge with the riprap from the structure tied into the bridge for protection. If the bridge is to be replaced, then it may be possible to construct the grade control structure concurrently with the road crossing. Figure 48 shows a combination grade control structure and road crossing.

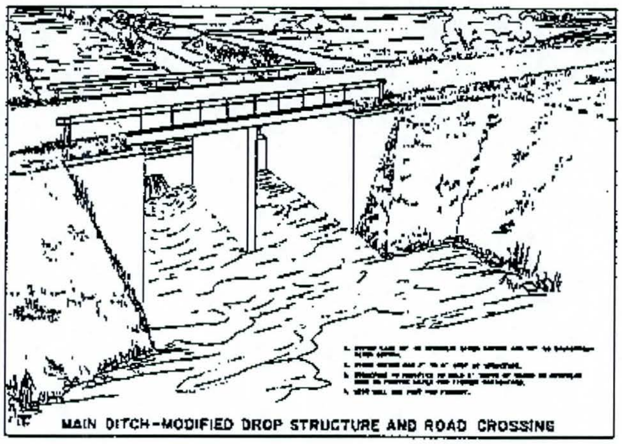


Figure 48. Combination grade control structure and road crossing (from <US Soil Conservation Service 1976>).

LOCAL SITE CONDITIONS

When planning grade control structures, the final siting is often adjusted to accommodate local site conditions, such as the plan form of the stream or local drainage. A stable upstream alignment that provides a straight approach into the structure is critical. Since failure to stabilize the upstream approach may lead to excessive scour and possible flanking of the structure, it is desirable to locate the structure in a straight reach. If this is not possible (as in the case in a very sinuous channel), it may be necessary to realign the channel to provide an adequate approach. Stabilization of the realigned channel may be required to ensure that the approach is maintained. Even if the structure is built in a straight reach, the possibility of upstream meanders migrating into the structure must be considered. In this case, the upstream meanders should be stabilized prior to, or concurrent with, the construction of the grade control structure.

Local inflows from tributaries, field drains, road side ditches, or other sources often play an important part in the siting of grade control structures. Failure to provide protection from local drainage can result in severe damage to a structure U.S. Army Corps of Engineers, 1981). During the initial siting of the structure, all local drainage should be identified. Ideally, the structure should be located to avoid local drainage problems. However, there may be some situations where this is not possible. In these instances, the local drainage should either be re-directed away from the structure or incorporated into the structure design in such a manner that there will be no damage to the structure. For example, on the Gering Drain in Nebraska, the CIT drop structures were modified to include interior drainage culverts and baffle walls to handle the local inflows (Stufft 1965).

DOWNSTREAM CHANNEL RESPONSE

Since grade control structures affect the sediment delivery to downstream reaches, it is necessary to consider the potential impacts to the downstream channel when grade control structures are planned. Bed control structures reduce the downstream sediment loading by preventing the erosion of the bed and banks, while hydraulic control structures have the added effect of trapping sediments. The ultimate response of the channel to the reduction in sediment supply will vary from site to site. In some instances the effects of grade control structures on sediment loading may be so small that downstream degradational problems may not be encountered. However, in some situations such as when a series of hydraulic control structures is planned, the cumulative effects of sediment trapping may become significant. In these instances, it may be necessary to modify the plan to reduce the amount of sediment being trapped or to consider placing additional grade control structures in the downstream reach to protect against the induced degradation.

GEOLOGIC CONTROLS

As previously mentioned, geologic controls often provide grade control in a similar manner to a bed control structure. In some cases a grade control structure can actually be eliminated from the plan if an existing geologic control can be utilized to provide a similar level of bed stability. However, caution must always be used when relying on geologic outcrops to provide long term grade control. In situations where geologic controls are to be used as permanent grade control structures, a detailed geotechnical investigation of the outcrop is needed to determine the vertical and lateral extent. This is necessary to ensure that the outcrop will neither be undermined nor flanked throughout the project life.

EFFECTS ON TRIBUTARIES

The effect of main stem structures on tributaries should be considered when siting grade control structures. As degradation on a main stem channel migrates upstream it will branch up into the tributaries streams. Therefore, the siting of grade control structures should consider effects on the tributaries. If possible, main stem structures should be placed downstream of tributary confluences. This will allow one structure to provide grade control to both the main stem and the tributary. This is generally a more cost effective procedure than having separate structures on each channel.

SUMMARY

The above discussion illustrates that the siting of grade control structures is not simply a hydraulic exercise. Rather, there are many other factors that must be included in the design process. For any specific situation, some or all of the factors discussed in this section may be critical elements in the final siting of grade control structures. It is recognized that this does not represent an all inclusive list since there may be other factors not discussed here that may be locally important. For example, in some cases, maintenance requirements, debris passage, ice conditions, or safety considerations may be controlling factors. Consequently, there is no definitive "cookbook" procedure for siting grade control structures that can be applied universally. Rather, each situation must be assessed on an individual basis.

TYPES OF GRADE CONTROL STRUCTURES

There are certain features which are common to most grade control structures. These include a control section for accomplishing the grade change, a section for energy dissipation, and protection of the upstream and downstream approaches. However, there is considerable variation in the design of these features. For example, a grade control structure may be constructed of riprap, concrete, sheet piling, treated lumber, soil cement, gabions, compacted earth fill, or other locally available material. Also, the shape (sloping or vertical drop) and dimensions of the structure can vary significantly, as can the various appurtenances (baffle plates, end sills, etc.). The applicability of a particular type of structure to any given situation depends upon a number of factors such as: hydrologic conditions, sediment size and loading, channel morphology, flood plain and valley characteristics, availability of construction materials, project objectives, and time and funding constraints. The successful use of a particular type of structure in one situation does not necessarily ensure it will be effective in another. Some of the more common types of grade control structures used in a variety of situations are discussed in the following sections.

SIMPLE BED CONTROL STRUCTURES

Perhaps the simplest form of a grade control structure consists of dumping rock, concrete rubble, or some other locally available non-erodible material across the channel to form a hard point. These structures are often referred to as rock sills, or bed sills. This is a very simple technique that is particularly popular with local landowners, small drainage districts or other agencies that do not have the resources to design and construct more elaborate structures. These type of structures are generally most effective in small stream applications and where the drop heights are generally less than about 2 to 3 feet. A series of rock sills, each creating a head loss of about two feet was used successfully on the Gering Drain in Nebraska (Stufft 1965). Figure 49 shows the Whitaker and Jaggi (Whittaker and Jaggi 1986) design concept for stabilizing the bed of a stream with a series of rock sills. The sills in Figure 49 are classic bed control structures that are simply acting as hard points to resist the erosion of the streambed.

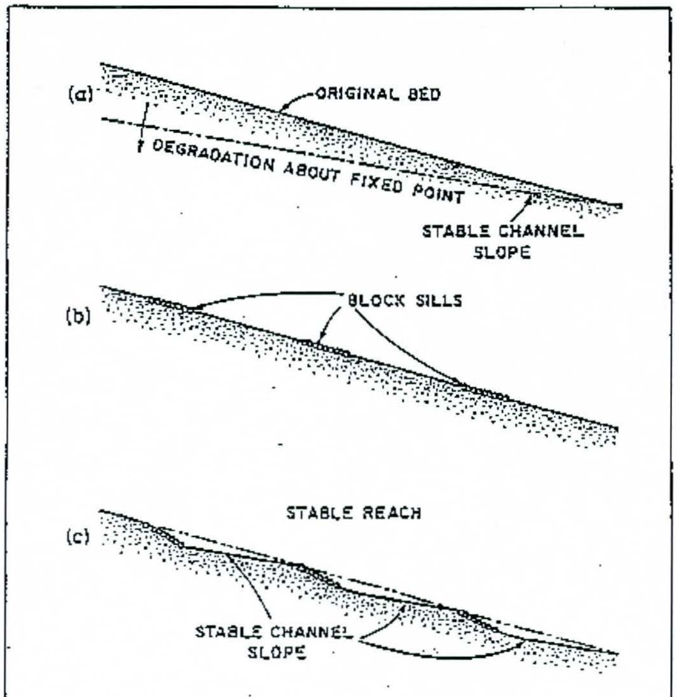


Figure 49. Channel stabilization with rock sills (from Whittaker and Jaggi 1986).

Construction of bed sills is sometimes accomplished by simply placing the rock along the streambed to act as a hard point to resist the erosive forces of the degradational zone. In other situations, a trench may be excavated across the streambed and then filled with rock. A critical component in the design of these structures is ensuring that there is sufficient volume of non-erodible material to resist the general bed degradation, as well as the local scour at the structure. Figure 50 and Figure 51 illustrate a riprap grade control structure designed to resist both the general bed degradation of the approaching knickpoint as well as any local scour that may be generated at the structure. In this instance, the riprap section must have sufficient mass to launch with an acceptable thickness to the anticipated scour hole depth. As a general rule, the launch slope is assumed to be slightly flatter, for a conservative design, than the natural angle of repose of the riprap. When designed properly, these structures can be an inexpensive, effective method of providing grade control. However, all too often, these structures are constructed hastily by individuals or groups not fully cognizant of the dynamics of stream behavior. When this occurs, the result is usually costly maintenance problems and often the complete failure of the structure.

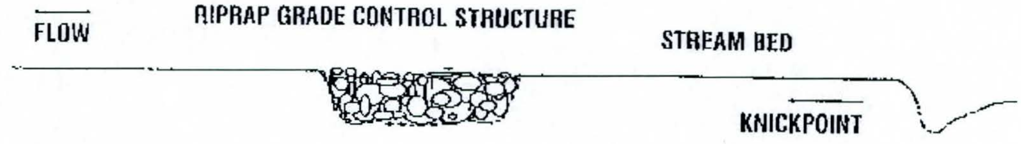


Figure 50. As-built riprap grade control structure with sufficient launch stone to handle anticipated scour.

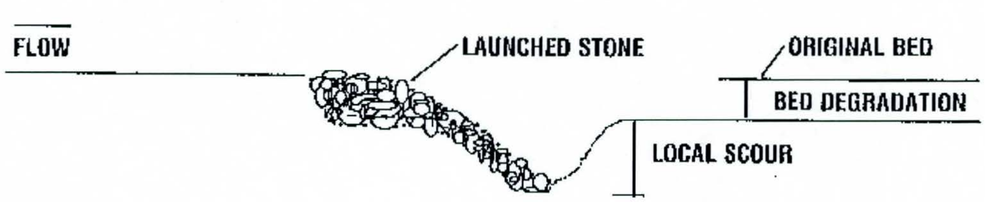


Figure 51. Launching of riprap at grade control structure in response to bed degradation and local scour.

STRUCTURES WITH WATER CUTOFF

One problem often encountered with the above structures is the displacement of rock (or rubble, etc.) due to the seepage flow around and beneath the structure. This is particularly a problem when the bed of the channel is composed primarily of pervious material. This problem can be eliminated by constructing a water barrier at the structure. One type of water barrier consists of simply placing a trench of impervious clay fill upstream of the weir crest. Figure 52 and Figure 53 illustrate this type of water barrier. One problem with this type of barrier is its longevity due to susceptibility to erosion. This problem can be avoided by using concrete or sheet piling for the cutoff wall. An additional advantage of having a sheet pile or concrete cutoff wall is that it serves as a dependable control section for establishing the upstream grade. Figure 54 and Figure 55 show the conceptual design of a riprap grade control structure with a sheet pile cutoff wall. In the case of the sloping riprap drop structures used by the Denver Urban Drainage and Flood Control District, an impervious clay fill is used in conjunction with a lateral cutoff wall (McLaughlin Water Engineers 1986). Figure 56 illustrates this design.

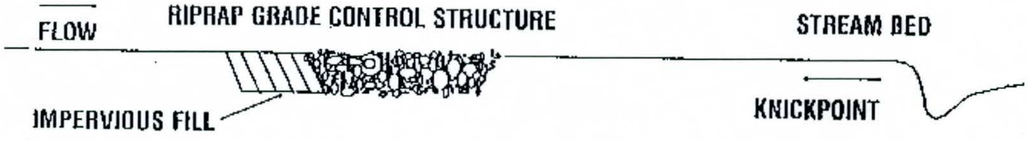


Figure 52. As-built riprap grade control sstructure with impervious fill cutoff wall.

Figure 53. Launching of riprap at grade control structure in response to bed degradation and local scour.

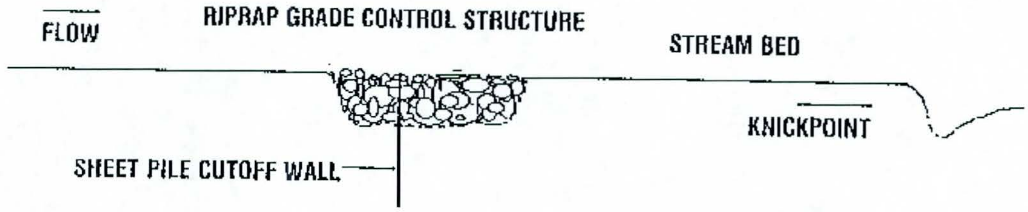


Figure 54. As-built riprap grade control structure with sheet pile cutoff wall.

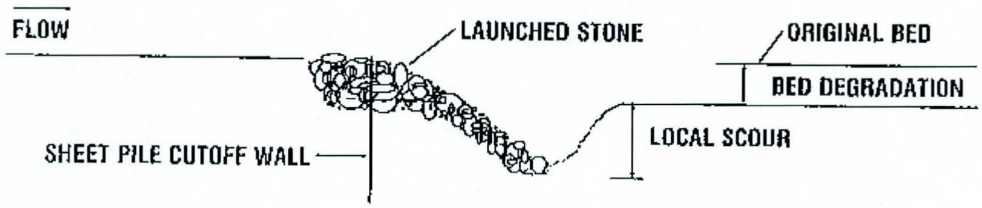


Figure 55. Launching of riprap at grade control structure in response to bed degradation and local scour.

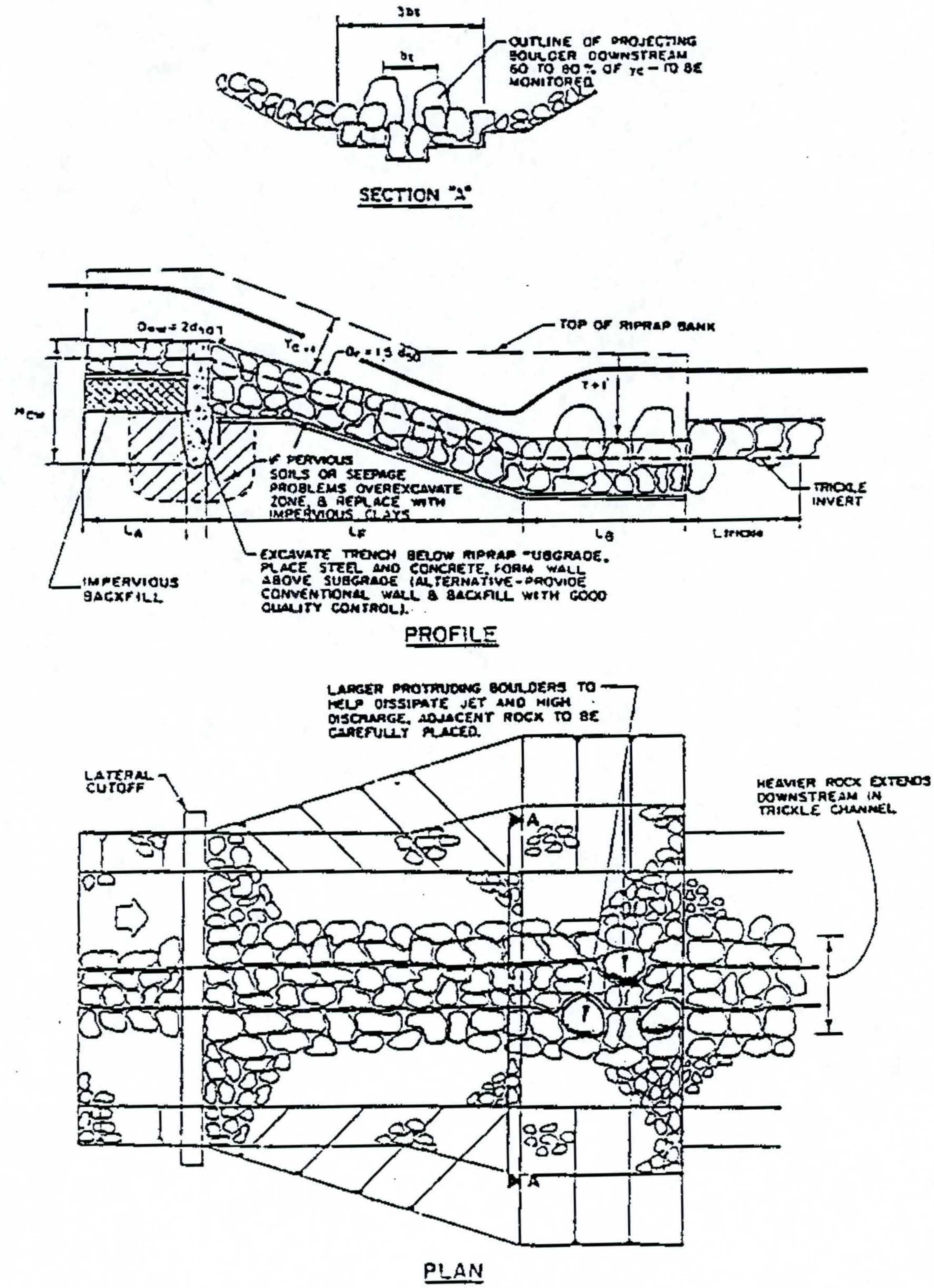


Figure 56. Sloping drop grade control structure with pre-formed riprap lined scour (from McLaughlin Water Engineers 1986).

STRUCTURES WITH PRE-FORMED SCOUR HOLES

A significant feature that distinguishes the sloping riprap structure of Figure 56 from the other structures discussed in the sections SIMPLE BED CONTROL STRUCTURES and STRUCTURES With WATER CUTOFF is the preformed, rock protected scour hole. A scour hole is a natural occurrence downstream of any drop whether it is a natural overfall or a man-made structure. As mentioned in the section SIMPLE BED CONTROL STRUCTURES a rock grade control structure must have sufficient launching rock to protect against the vertical scour immediately below the weir section. However, the lateral extent of the scour hole must also be considered to ensure that it does not become so large that the structure is subject to being flanked. With many simple grade control structures in small stream applications, very little, if any attention is given to the design of a stilling basin or pre-formed scour hole, but rather, the erosion is allowed to form the scour hole. However, at higher flow and drop situations, a preformed scour hole protected with concrete, riprap, or some other erosion resistant materials is usually warranted. This scour hole serves as a stilling basin for dissipating the energy of the plunging flow. Sizing of the scour hole is a critical element in the design process which is usually based on model studies or on experience with similar structures in the area. If the scour hole is too large, there is often a tendency for sediment to deposit along the perimeter of the scour hole. If it is too small, then there may be a potential for failure of the rock due to the scouring forces of the water.

The stability of rock structures is often jeopardized at low tailwater conditions due to the stability of the rock, which is often the limiting factor in determining the maximum drop height of the structure. One way to ensure the stability of the rock is to design the structure to operate in a submerged condition. Figure 57 shows the Corps basis for design of such a bed stabilizer (U.S. Army Corps of Engineers 1970). Model studies were performed as part of the Floyd River Flood Control Project in Sioux City Iowa where a series of these structures were designed to prevent channel degradation. The model studies indicated that these structures would perform satisfactorily as long as they were designed to operate at submerged conditions where the tailwater (T') did not fall below 0.8 of the critical depth (D0) at the crest section (Linder 1963). Subsequent monitoring of the in place structures confirmed their successful performance in the field (U.S. Army Corps of Engineers 1981).

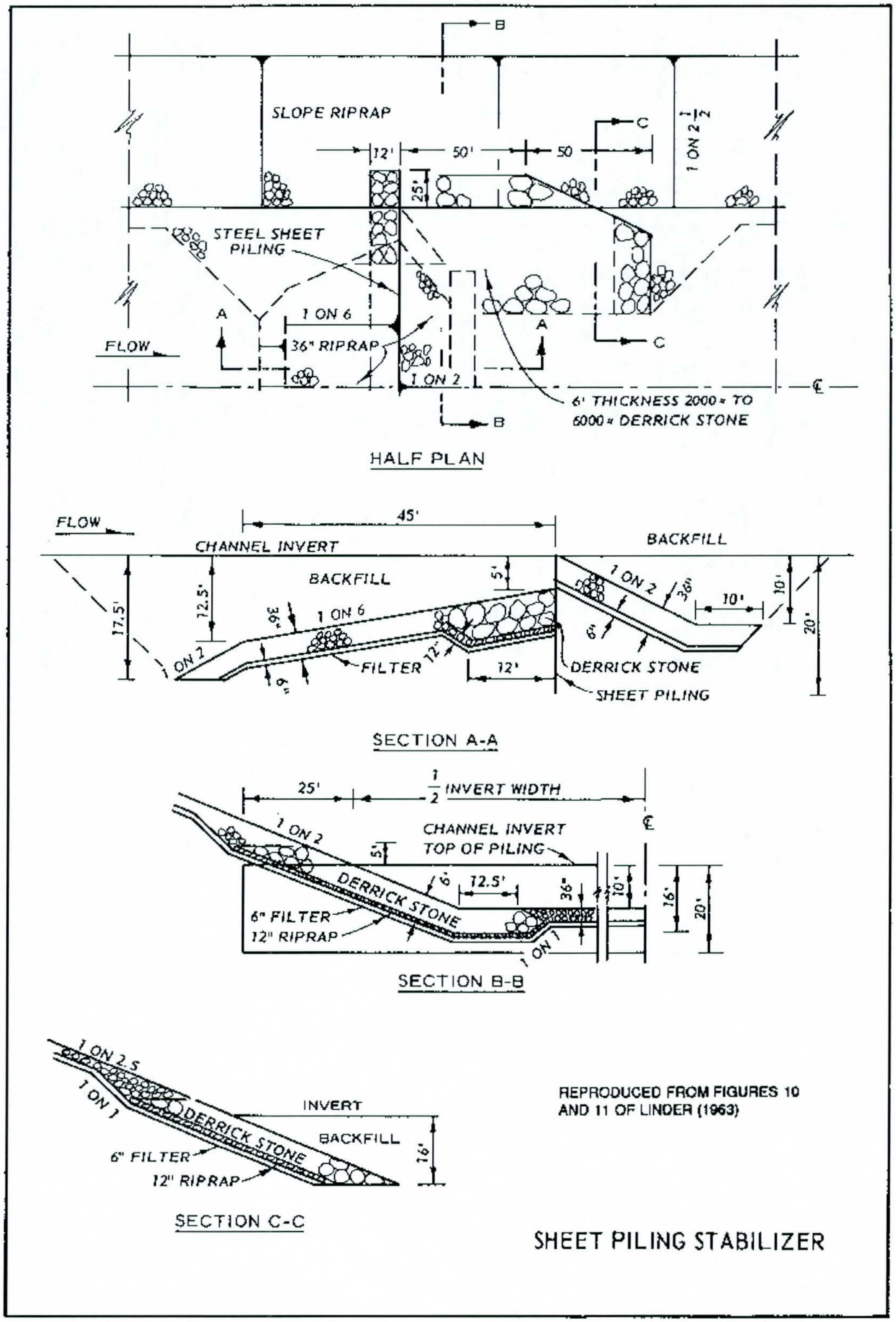


Figure 57. Bed stabilizer design with sheet pile cutoff (<US Army Corps of Engineers 1970>Plate B-45).

In many instances, the energy dissipation in a grade control structure is accomplished by the plunging action of the flow into the riprap protected stilling basin. This is generally satisfactory where the degree of submergence is relatively high due to small drop heights and/or high tailwater conditions. However, at lower submergence conditions where drop heights are large or tailwater is low, some additional means of dissipating the energy must be provided. Little and Murphy (Little and Murphy 1982) observed that an undular hydraulic jump occurs when the incoming Froude number is less than 1.7. If this condition exists, very little energy is dissipated resulting in the potential for failure of the riprap in the stilling basin and for increased downstream bed and bank erosion due to the downstream progression of undular waves. Consequently, Little and Murphy developed a grade control design that included an energy dissipating baffle to break up these undular waves. Figure 58 shows the plans for this type of grade control structure. This structure, the ARS type low drop structure is in use successfully in North Mississippi for drop heights up to about six feet by both the U.S. Army Corps of Engineers and the Soil Conservation Service (U.S. Army Corps of Engineers 1981). One problem encountered with the ARS structures has been the stability of the riprap in the stilling basin. This prompted model studies at Colorado State University to modify the structure and reduce the riprap size necessary for stability (Johns et al. 1993, and Abt et al. 1994). Figure 59 is a schematic of the modified ARS structure. The modified structure retains the baffle plate and adopts a vertical drop at the sheet pile rather than a sloping rock-fill section.

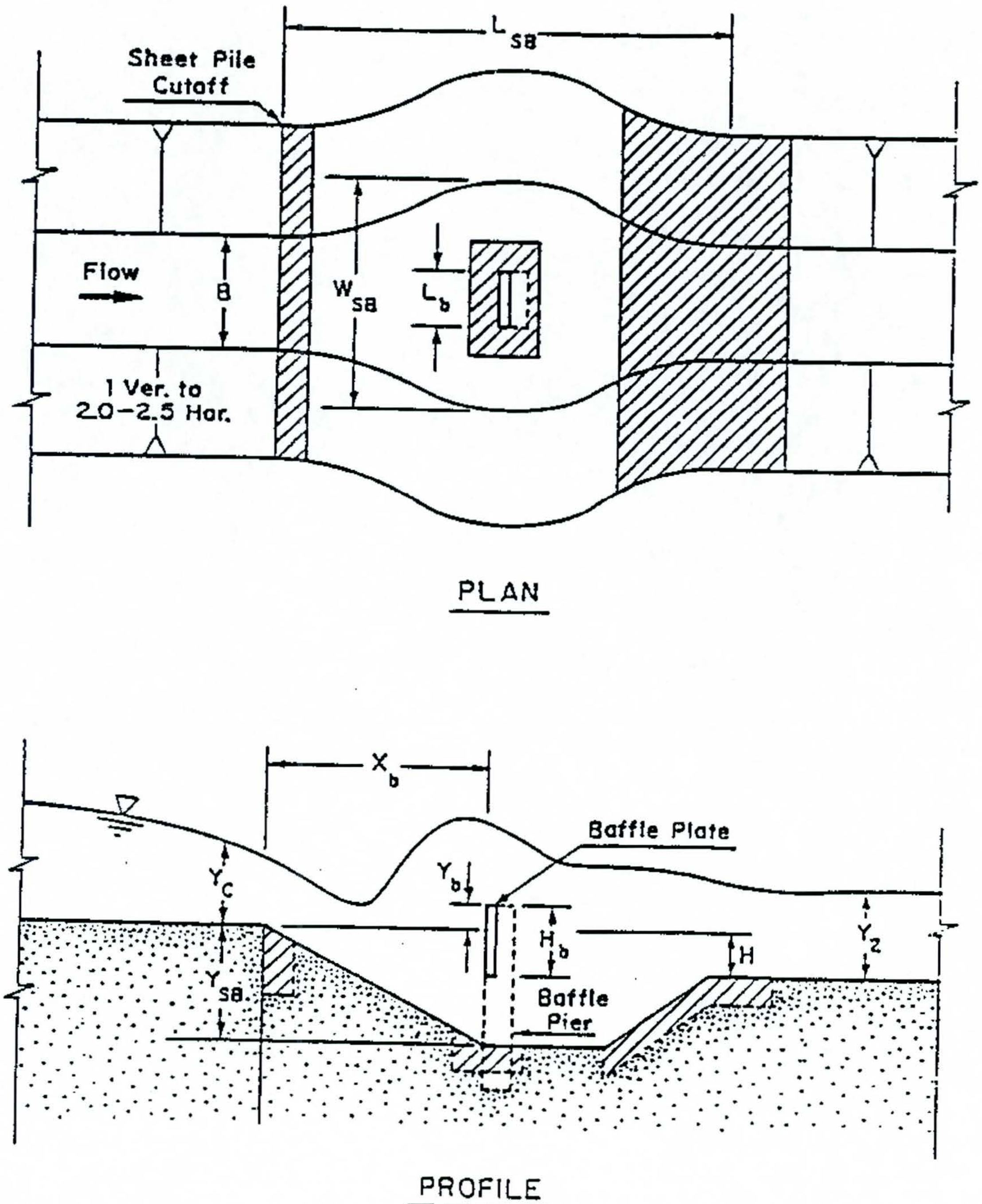


Figure 58. ARS-Type grade control structure with pre-formed riprap lined stilling basin and baffle plate (Little and Murphy 1982).

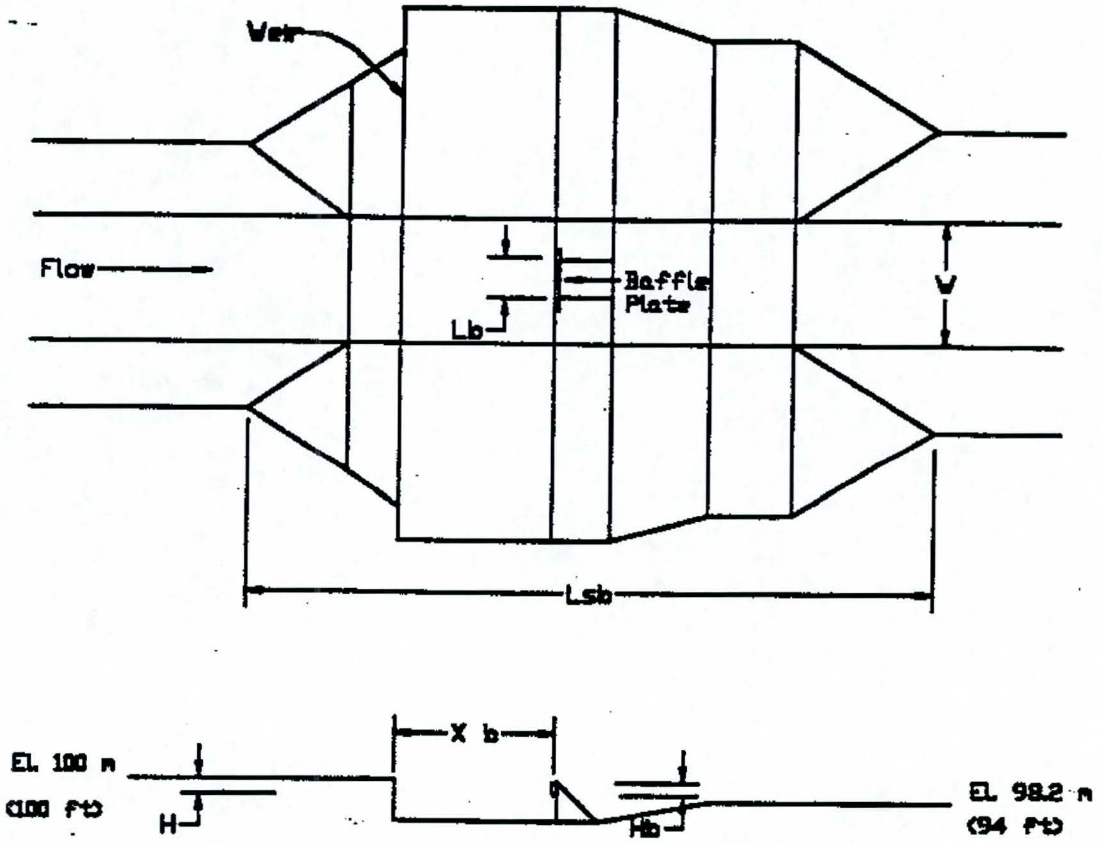


Figure 59. Schematic of modified ARS-Type grade control structure (Abt et al. 1994).

CONCRETE DROP STRUCTURES

In many situations where the discharges and/or drop heights are large, grade control structures are normally constructed of concrete. There are many different designs for concrete grade control structures. The two discussed herein are the CIT and the St. Anthony Falls (SAF) structures. Both of these structures were utilized on the Gering Drain project in Nebraska, where the decision to use one or the other was based on the flow and channel conditions (Stufft 1965). where the discharges were large and the channel depth was relatively shallow, the CIT type of drop structure was utilized. The CIT structure is generally applicable to low drop situations where the ratio of the drop height to critical depth is less than one; however, for the Gering Drain project this ratio was extended up to 1.2. The original design of this structure was based on criteria developed by Vanoni and Pollack (Vanoni and Pollack 1959). The structure was then modified by model studies at the Waterways Experiment Station in Vicksburg MS. Figure 60 shows the CIT-Type drop structure. (Murphy 1967a). Where the channel was relatively deep and the discharges smaller, the SAF drop structure was used. This design was developed from model studies at the Saint Anthony Falls Hydraulic Laboratory for the U.S. Soil Conservation Service (Blaisdell 1948). Figure 61 shows the Saint Anthony Falls type structure. The SAF structure is capable of functioning in flow situations where the drop height to critical depth ratio is greater than one and can provide effective energy dissipation within a Froude number range of 1.7 to 17. Both the CIT and the SAF drop structures have performed satisfactorily on the Gering Drain for over 25 years.

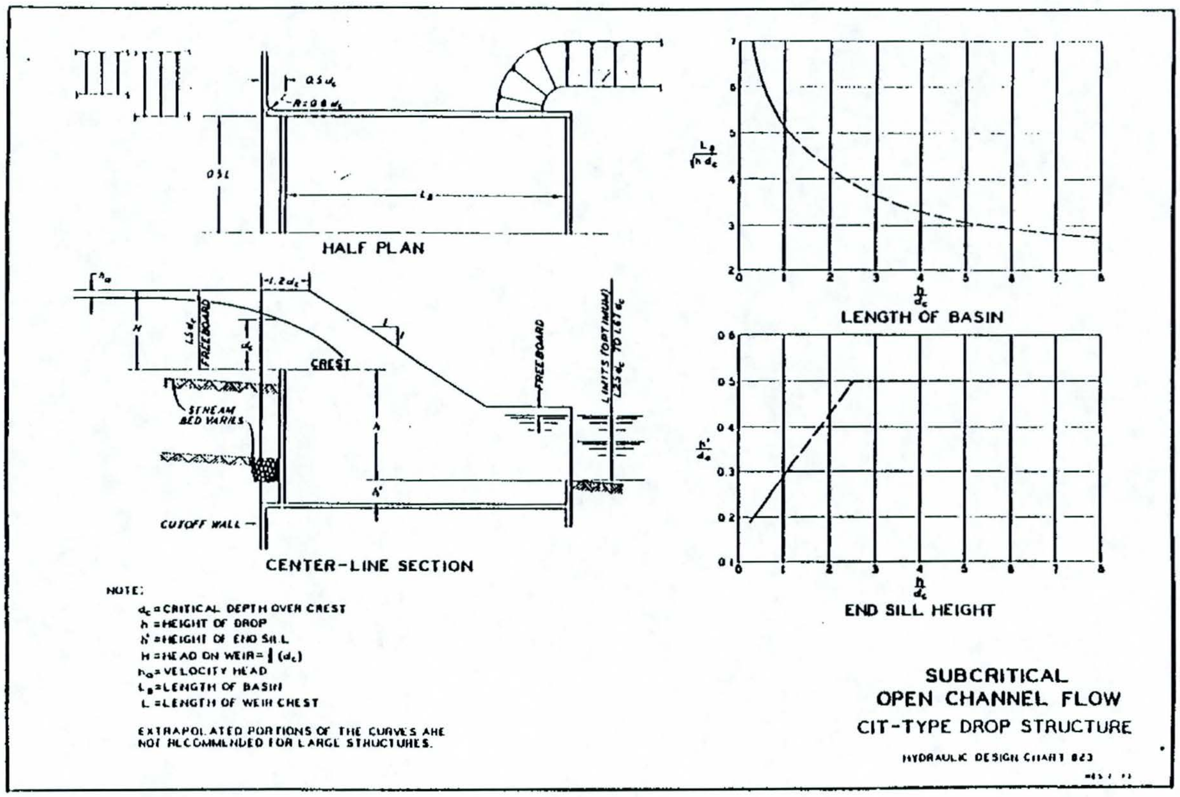


Figure 60. CIT-Type drop structure (Murphy 1967a).

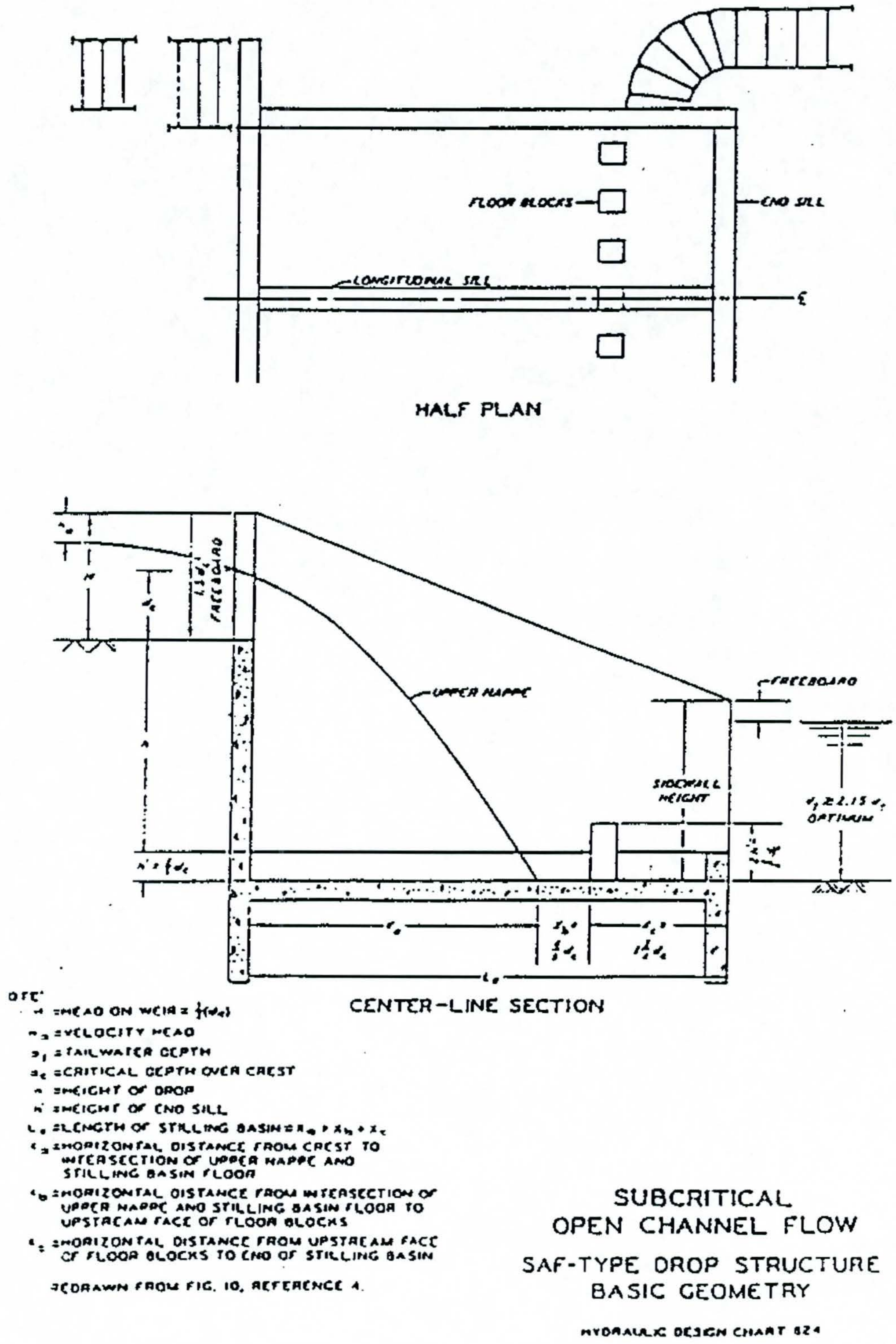


Figure 61. Saint Anthony Falls (SAF) type drop structure (Blaisdell 1948).

CHANNEL LININGS

Grade control can also be accomplished by lining the channel bed with a non-erodible material. This technique differs somewhat from the previous examples in that the drop is effected over a relatively long reach of the channel rather than at a localized control section. These structures are designed to ensure that the drop is accomplished over a specified reach of the channel which has been lined with riprap or some other non-erodible material. Rock riprap gradient control structures have been used by the U.S. Soil Conservation Service for several years (U.S. Soil Conservation Service 1976). These structures are designed to flow in the subcritical regime with a constant specific energy at the design discharge which is equal to the specific energy of flow immediately upstream of the structure (Myers 1982). Although these structures have generally been successful, there have been some associated local scour problems. This precipitated a series of model studies at the Waterways Experiment Station to correct these problems and to develop a design methodology for these structures (Tate 1987 and Tate 1991). Figure 62 shows a plan and profile drawing of the improved structure.

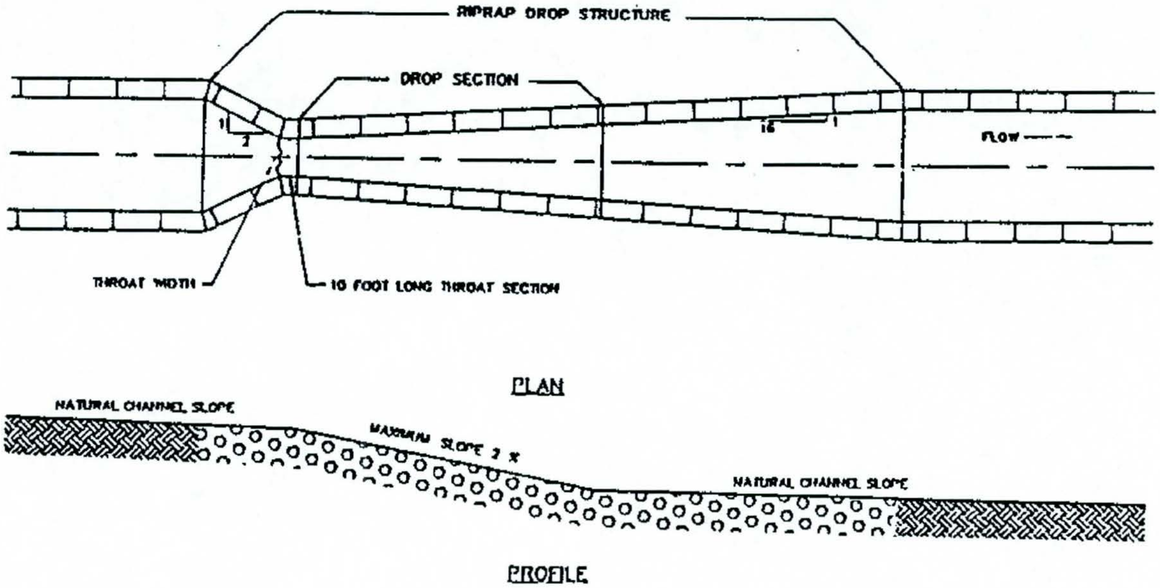


Figure 62. Riprap lined drop structure (Tate 1991).

ALTERNATIVE CONSTRUCTION MATERIALS

While riprap and concrete may be the most commonly used construction materials for grade control structures, many situations where cost or availability of materials may prompt the engineer to consider other alternatives. In these situations it may be more cost effective to utilize locally available materials. Gabion grade control structures are often an effective alternative to the standard riprap or concrete structures (Hanson et al. 1986). Agostini et al. 1988 provides design criteria for vertical, stepped, and sloped type gabion grade control structures, as well as examples of completed works. Guidance for the construction of Gabion weirs is also provided by the Corps of Engineers' ETL 1110-2-194 (U.S. Army Corps of Engineers 1974).

Another alternative to the conventional riprap or concrete structure which has gained popularity in the southwestern U.S. is the use of soil cement grade control structures. These structures are constructed of on site soil-sand in a mix with Portland Cement to form a high quality, erosion resistant mixture (Simons, Li and Associates 1981). Soil cement grade control structures are most applicable when used as a series of small drops in lieu of a single large drop structure (Gemma et al. 1982). Experience has indicated that a limiting drop height for these structures is on the order of three feet. Design criteria for these structures is presented by Simons, Li and Associates 1981.

SUMMARY

The concept of grade control is extremely important in river engineering projects. Theoretically, grade control can be accomplished by adjusting the flow or sediment loading, but the most commonly used practice is to construct in-channel grade control structures. In-channel structures are classified as either bed control structures, or hydraulic control structures depending upon the process by which the grade is controlled. Bed control structures function by creating a hard point in the bed that is capable of resisting the erosional forces of the degradational zone. Hydraulic control structures function by reducing the energy slope upstream in the degradational zone.

Perhaps the most important aspect in the design of grade control structures is the proper siting of the structure within the channel system to meet the project objectives. The procedure for siting grade control structures is often considered to be a simple optimization of hydraulics and economics. However, in practice these factors alone are usually not sufficient to define the optimum siting conditions for grade control structures. Rather, the geotechnical stability of the stream banks, flood control impacts, environmental considerations, existing structures, local site conditions, potential downstream channel response, geologic controls, effects on tributaries, and many other factors must be integrated with the hydraulics and economics to select the best plan for the project area.

There are literally hundreds of different designs of grade control structures used world-wide, none of which is universally applicable to all situations. The grade control structures outlines herein were presented with the intent of encompassing a wide range of structure types used in various design conditions. These structures represent only a small fraction of the many different types of designs that are commonly used. For more information on various structure designs, the reader is referred to Neilson (Neilson et al. 1991), which provides a comprehensive international literature review on grade control structures with an annotated bibliography.

Grade control structures have been used effectively as erosion control features in water resources projects for many years. Unfortunately, these structures are often considered rehabilitative features to be used only after something has gone wrong with the system. However, a more effective use of these structures is to incorporate them into the initial plans for the channel project in a proactive rather than reactive mode. As water resource projects become more and more complex, there is an increasing need for grade control structures to be utilized in a much broader sense to provide for environmental sustainability as well as erosion control.

<The following pages are a paper by Wittler for the 96 ASCE conference.>

SITING

This section describes the process of siting grade control structures based upon a water surface profile of the reach and a site reconnaissance. Siting criteria include discontinuities in the low-water surface profile, identification of a stable reach of the creek for emulation, access for construction, and economics of the project.

STABLE CHANNEL SLOPE

Traditional grade control schemes use the "stair step" concept. This concept dictates that the crest of a downstream structure must be higher than the invert of the upstream structure. The primary assumption of this concept is a near-horizontal water surface between the structures. The upstream-downstream relationship between grade control structures prevents general degradation in the channel between the structures. A factor that modifies the stair step concept is the stable channel slope of the stream between the structures. If the structures are far enough apart that the channel partially or fully controls the hydraulics of the stream, then the slope of the water surface is a factor in the stair step scheme.

When the grade control structures totally control the hydraulics, then the water surface is near-horizontal between structures. As the distance and or flow between structures increases the channel control increases, and the slope of the water surface increases. Figure 63 shows the difference between a near-horizontal water surface and a sloped water surface. In this example, increasing the water surface slope from near-horizontal to 0.0192 increases the spacing between the structures by fifty percent. Increasing the spacing between the structures decreases the number of structures required to control the grade.

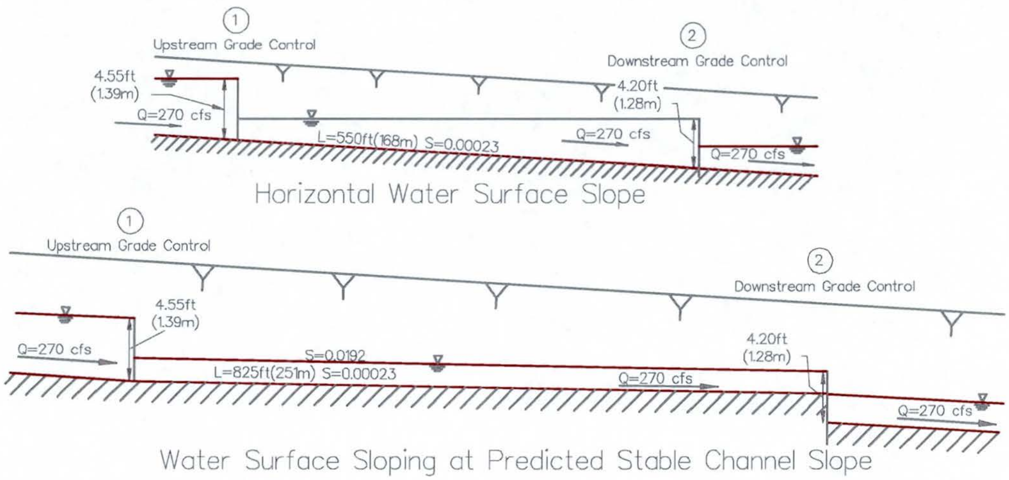


Figure 63. Stair step schemes with near-horizontal and sloped water surfaces between structures. Sloping the water surface 0.0192 increases the spacing by 50%.

The key to proper spacing, or siting, of multiple grade control structures is an accurate prediction of the water surface slope between each pair of structures. The simplest prediction is a near-horizontal water surface. However this very rarely occurs, usually in channels with very high conveyance capacity and low discharge. The factors that govern the prediction of water surface slope include discharge, wetted perimeter, hydraulic radius, bed and bank roughness, sediment transport, bed slope, and local features such as natural outcroppings, hard points, or islands.

Installing low profile grade control structures simplifies the problem of predicting the stable channel slope. By creating relatively short pools, and minimizing in-stream storage of sediments, the impacts upon the hydraulics of a controlled stream are minimal. In this case, the best estimate of the stable channel slope may be determined by analyzing a relatively stable reach of the stream, or another similar stream. Analytical methods for predicting the stable channel slope are in development by Raphelt (Raphelt et al. 1995).

TECHNICAL AND SUBJECTIVE SITING DATA

After ascertaining the stable channel slope, an analysis of the water surface and thalweg profiles provides the final data necessary for siting grade control structures. Changes in the grade indicate areas where installation of grade control structures will be most effective. Combining water surface and thalweg profiles with site reconnaissance is a satisfactory method of siting grade control structures. The site reconnaissance also provides the opportunity to assess and adjust to construction barriers prior to construction. The problem with this method of siting is the choice that inevitably arises. Without a precise estimate of the stable channel slope, the structures may be placed too far apart or too close together. Economics require the least number of structures. A high estimate of the stable channel slope results in fewer grade control structures. However, if the spacing is overestimated, then general degradation will occur between structures, destabilizing the upstream structures. The designer has the choice of installing too few structures, risking destabilization, or installing too many structures, abusing the economics of the restoration project, but ensuring the stability of the grade control scheme.

WATER SURFACE AND THALWEG PROFILES

Figure 64 shows the water surface profile of the Muddy Creek Phase II reach. The 4-mile reach was surveyed in July 1995, during normal flow over a two day period. The survey has been adjusted for changes in stage. The plot shows two distinct grades. The first is in the lower portion of the reach, and the second is in the upper portion of the reach. The vertical offset between the two grades is roughly 0.8 feet.

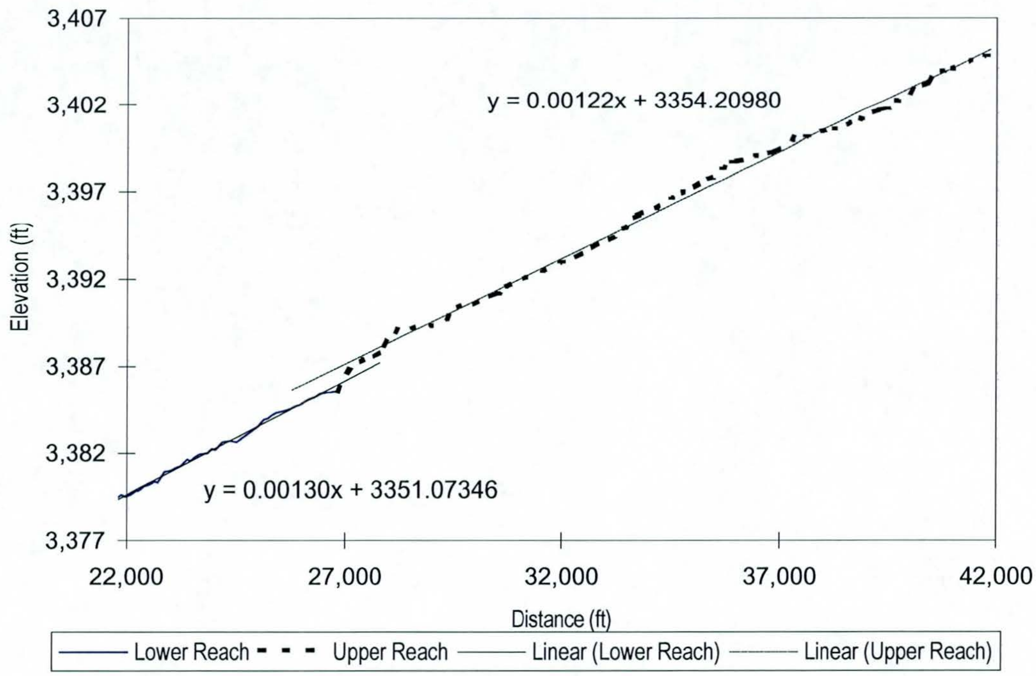


Figure 64. Water surface profile of Muddy Creek Phase II reach.

Figure 65 shows 11,000 feet of the Phase II reach, adding contrast to the vertical offset between the two reaches of the stream. The lower reach is steeper on average, 0.0013 versus 0.0012 in the upper reach. The change in grade occurs between 27,000 and 29,000 feet upstream from the start of the project. This data led to the installation of two low profile chevron weir rock ramps in December 1995. Each grade control structure has a design hydraulic drop of 1.5 feet, for a total of 3.0 feet of hydraulic drop. The two structures were installed roughly 1,000 feet apart starting at 26,000 feet. The stable channel slope of the lower reach is 0.0012, based upon the remainder of the upper portion of the Phase II reach.

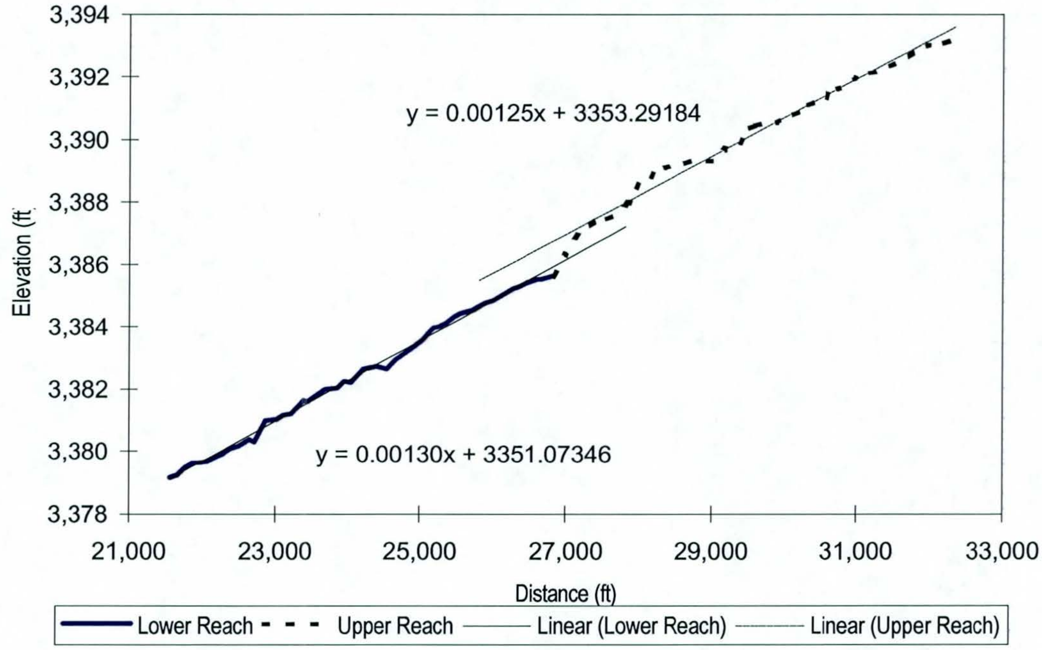


Figure 65. Water Surface profile of short portion of Muddy Creek Phase II reach.

Figure 66 shows the corresponding thalweg profile of the Muddy Creek Phase II reach. The thalweg in this case was defined to be the lowest point on the bed at each survey station. The rod operator searched the bottom of the channel until the approximate low point was found. The spacing between survey points was roughly 50 feet. The figure demonstrates the greater usefulness of the water surface profile in this case. The variability of the thalweg is unsuitable for determining subtle changes in grade.

Keys for proper utilization of a water surface profile for siting grade control structures are:

- Frequent measurements of the water surface elevation along the reach.
- Measurements collected during stable periods of flow adjusting for stage changes.
- Measurements collected during a representative discharge, or a discharge selected for design purposes.
- Proper interpretation of island effects, hard points, riffles, and other local hydraulic effects.
- Integration with thalweg data, site reconnaissance data, and construction preferences.

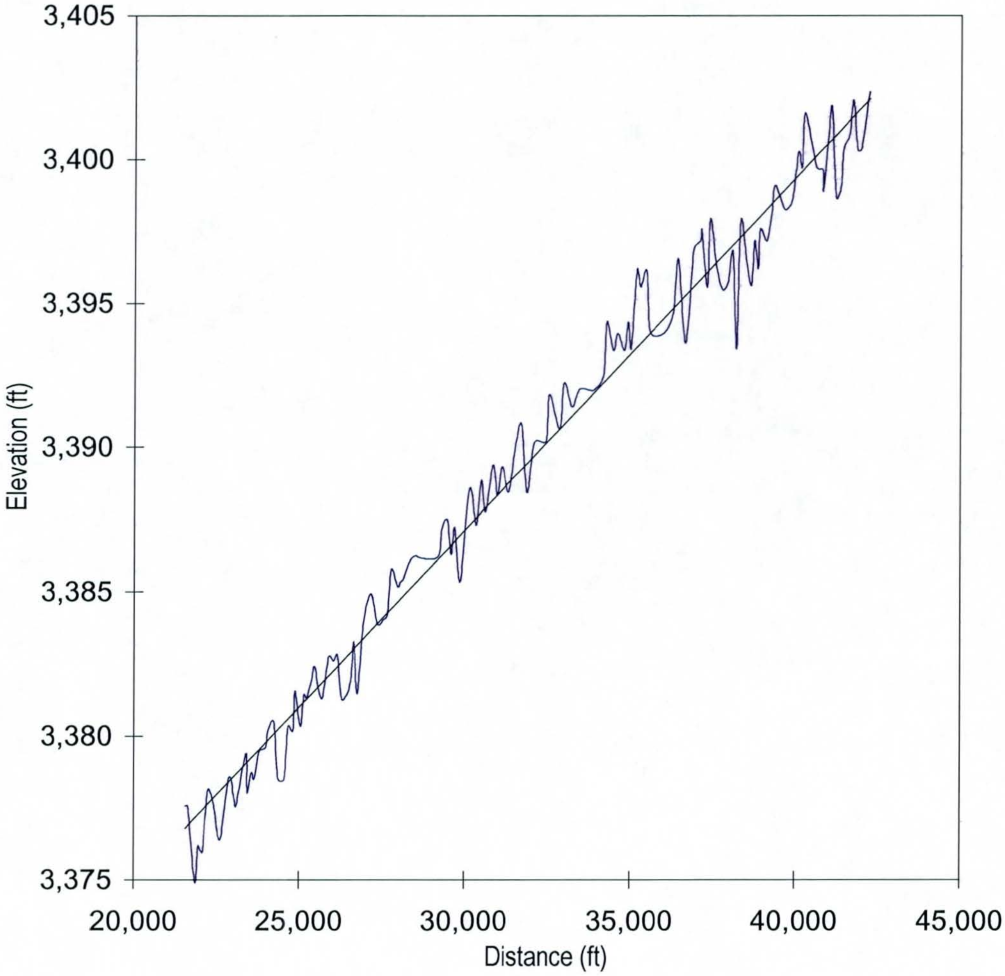


Figure 66. Thalweg profile.

SITE RECONNAISSANCE

Although grade control structure location can be a function of the water surface profile only, a site reconnaissance should follow the theoretical siting. The reconnaissance gives the designer the opportunity to use the keys to successful interpretation of the water surface profile, such as local hydraulic effects and a general feel for the stream.

ECONOMICS: MANY OR FEW?

The economics of grade control are very dependent upon the stable channel slope and thus the longitudinal frequency of grade control structures. The choice, mentioned earlier, is to install too many or too few structures, depending upon the level of risk the designer is willing to bear. Maintenance can mitigate the risk, and is therefore also a crucial factor in the economics. With a strong commitment to maintenance of grade control structures, the designer may add risk and place structures farther apart. If the performance of the grade control does not meet expectations then additional structures may be necessary, or aggressive maintenance of the existing structures is necessary. At Muddy Creek, the Task Force —a local group directing the stream restoration project —chose an economical estimate of the stable channel slope, and spaced the structures relatively far apart. Since construction of the original eight grade control structures it has been necessary to add a ninth structure between the fifth and sixth structures. The sixth structure was experiencing severe local scour at the toe, scour exceeding 10 feet in depth. The design hydraulic height of the sixth structure was 1.75 feet, but after installation was roughly 2.75 feet. An intermediate structure was installed 500 feet downstream of the sixth structure. Installation of the intermediate structure reduced the hydraulic height of the sixth structure to roughly 1.5 feet.

WHERE THEY SHOULD BE AND WHERE THEY ARE

The water surface profile provides the initial locations for a series of grade control structures. During the site reconnaissance, final locations should be determined. The factors that influence the final siting include hydraulic and construction considerations. In general, grade control structures should be installed below local hydraulic features such as headcuts, and above islands, or natural hard points. Islands and natural hard points enhance the hydraulic stability of grade control structures by adding nonstructural height, effectively increasing spacing of the structures. A structure installed above a headcut is susceptible to undermining as the headcut advances into the structure.

Various contractors will utilize a wide variety of construction approaches. Familiarity with potential contractors and their methods prior to site reconnaissance gives the designer an insight into the suitability of final locations with respect to construction barriers.

- Since the economics of stream restoration projects using grade control are very dependent upon accurate prediction of the stable channel slope, researchers should increase the effort to develop analytical tools that aid the prediction.

- A well-executed survey of the water surface profile provides enough data for a preliminary siting of a series of grade control structures.

- A thalweg profile is not as useful as a water surface profile due to the increased subjectivity of defining the thalweg.

- A site reconnaissance is a requirement for final siting of grade control structures, taking into account local hydraulic effects and construction barriers.

>

SPACING

HORIZONTAL WATER SURFACE

<This method is the simplest and most conservative, from a number of structures standpoint. Might work for slow moving, small discharge, relatively flat applications. It serves as a lower bound for spacing distance between structures.>

RATING CURVE MATCH

<This method matches the rating of the grade control structure with that of the channel upstream of the installation. The purpose is to maintain the present water surface profile or to invoke either an M-1 or M-2 profile. In the context of "siting" relocating a structure to a more suitable location in order to match a rating curve may make sense. This is related to construction and local siting issues. You may have a nearby location that is more suitable, i.e. economical, constructable, etc., that will make it easier to match an upstream rating.>

REFERENCE REACH

<This method designates the stable channel slope as equal to a reference reach. There may be a relatively stable or undisturbed reach on the same stream that indicates the stable channel slope.>

LOCAL OR REGIONAL SLOPE -VS- DRAINAGE AREA

SHEAR STRESS

<Chester said something about Lane's relationships>

STABLE ALLUVIAL CHANNEL DESIGN

<This method is the "SAM" method. SAM and other models like it, simultaneously solve the hydraulic and sediment transport conditions in a stream. The results yield a family of channel dimensions that based upon the hydraulics will transport sediments through a section at the same rate sediments enter the section.>

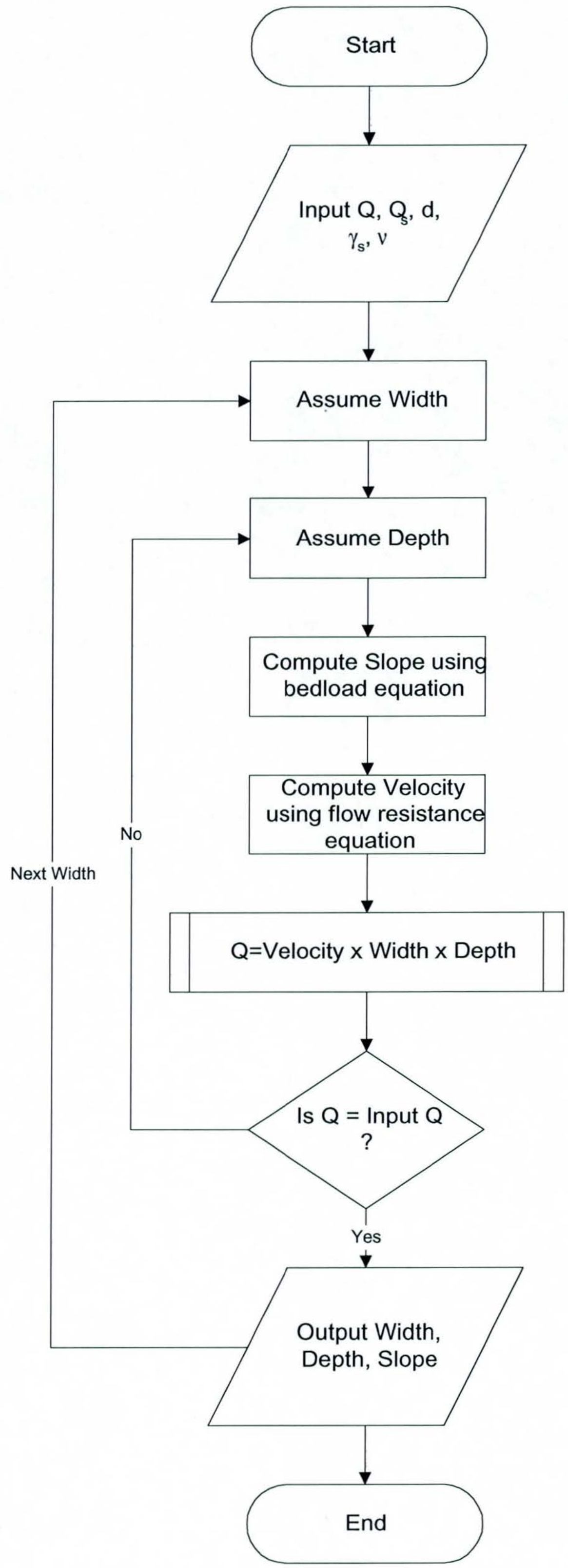


Figure 67. Stable channel design flowchart.

MONITORING & MAINTENANCE

<The following section is from "Monitoring of DEC Drop Structures" by Watson, Abt, & Gessler.>

<This section needs an introduction setting up the case study of the DEC project. We need definitions of monitoring and maintenance. We need basic criteria for both, along with objectives of each. Emphasize that the purpose is to learn and optimize resources while controlling the grade of important waterways.>

CASE STUDY –DEMONSTRATION EROSION CONTROL PROJECT

The Demonstration Erosion Control (DEC) Project provides for the development of a system for control of sediment, erosion, and flooding in the foothills area of the Yazoo Basin, Mississippi. Structural features that are used in developing rehabilitation plans for the DEC watersheds include: high-drop grade control structures similar to the SCS Type-C structure, drop box culverts, and low-drop grade control structures similar to the ARS low-drop structure.

Watson (Watson et al. 1988) reported on the evaluation of channel response for several Yazoo Basin low drop structures. This evaluation was based on comparison of then-current structure surveys with available historic information. Although this data base was limited, several recommendations were presented; design procedures for the low-drop structure must incorporate tailwater definition, upstream aggradation was limited due to the lack of constriction at the weir, little test data existed for operation of the structures at high submergence, and structures should be planned based on a comprehensive watershed stabilization plan. Some of these recommendations have resulted in additional physical modeling to supplement the excellent work by Little and Murphy (Little and Murphy 1981, Little and Murphy 1982). A series of hydraulic model tests has been conducted at Colorado State University by Abt et al. (Abt et al. 1990, Abt et al. 1992) to evaluate the low-drop structure under conditions of flow that were not considered by Little and Murphy (Little and Murphy 1981, Little and Murphy 1982). Results from the CSU tests indicated that riprap stability in many existing structures was poor, and field confirmation of the riprap instability has been documented by Lenzotti and Fullerton (Lenzotti 1990). Watson et al. (Watson et al. 1995) reported on the 1993 field inspection of the DEC grade control structures. Monitoring of the structures was conducted to reinforce previous grade control physical modeling studies and analyses conducted by Waterways Experiment Station (WES) and Colorado State University (CSU). Six common problems were observed in the inspection of approximately 60 structures, and recommendations were made for maintenance and restoration activities in that publication.

As a result of the studies cited, and continuing efforts to improve the structure design and application by Vicksburg District personnel, the drop structure design has changed several times. Each change has improved performance for the subsequent structures. This report documents the second opportunity that resources and personnel have been available to make a field evaluation of each structure and to make restoration recommendations that may be required.

OBJECTIVE

The objective of this applied research is to document the condition of U.S. Army Corps of Engineers high-drop and low-drop grade control structures constructed in the Yazoo Basin as a part of the Demonstration Erosion Control (DEC) program, and to make recommendations for restoration of these structures if necessary. A general objective of this research is to contribute to the development of improvements in the general design of grade control structures, and to compare the various types and ages of structures to develop a database that may be useful in predicting restoration needs and design improvements for similar structures.

REPORT ORGANIZATION

Chapter 2 describes the procedures used in the site visits and the results generated. A field evaluation form was completed, a brief description of the channel conditions upstream and downstream, and a thalweg profile for each structure visited is included in Appendix A Structures under construction at the time of field evaluations were not included. Chapter 3 includes the results of the investigation and Chapter 4 is a summary of the evaluations and conclusions that can be drawn. Each structure evaluation form includes a film roll number and slide numbers for those slides pertaining to that structure. All text and slides will be available in a separate magnetic volume that is being produced on CD-ROM.

METHODOLOGY

Each structure site was visited by Drs. Abt and Watson, and the thalweg was surveyed by CSU personnel. A one-page evaluation form was prepared for each structure, which includes the stream name and structure number, the date of the field evaluation, and the type of structure. Comments pertaining to rock stability, debris accumulation, vegetation, and other factors are recorded or eight locations within each drop structure. The form includes a location for comments on debris and vegetation generally related to the structure, and a location for comments concerning overbank drainage. Recommendations for restoration alternatives are presented in narrative form at the bottom of the form. The final data supplied on the form is the film roll number and a series of slide numbers that pertain to photographic documentation taken in the field.

In addition to the form for each structure, a narrative discussion of each stream reach was prepared, and is contained in the Appendix a along with the forms and thalweg profiles. This report includes all data collected except slides. A CD-ROM copy of this report is being prepared and will include access to all the slide images taken during the field investigations.

RESULTS

Although each structure operates under unique conditions, many similarities exist and some of the problems that were observed can be summarized. Table 1 lists the eight most common problems observed and Table 2 compares the 1993 and 1995 results.

Table 1. Common Low Drop Structure Problems.

1. Riprap is displaced from the face of the weir.
2. The channel bank upstream or downstream of the structure fails.
3. Bank erosion or piping beneath the riprap that is caused by overbank drainage.
4. Riprap is launching at the upstream or downstream apron.
5. Severe head cutting is migrating into the basin.
6. Woody vegetation has become established in the upstream or downstream apron, and is impairing the conveyance or the weir unit discharge of the structure.
7. Active incision is present downstream of the structure.
8. The thalweg upstream of the structure is below the weir crest for more than 500 feet.

Table 2. Comparison of 1993 and 1995 Frequency of Problems.

Problem	1	2	3	4	5	6	7	8
1993	41%	37%	24%	28%	17%	19%		
1995	37%	23%	18%	43%	37%	11%	69%	78%

In addition to identification of the types of problems and recommendations for resolving these problems, each structure has been assigned a prioritization category:

* Category 1 structures are under an imminent threat of loss of function;

* Category 2 structures have problems that should be resolved;

* Category 3 structures have no significant problems.

Table 3 is a summary of the eight problems previously defined and the category for each structure evaluated. Five structures were in category 1, 32 in category 2, and 18 in category 3 for the 1993 inspection. In the 1995 inspection, only three structures were in category 1, with all of the 1993 category 1 structures having been rehabilitated. Only 12 structures were in category 2, representing a decrease in category 2 of twenty structures. The remaining 50 structures inspected in 1995 are category 3 structures. The eight problems are addressed by either a 1 or 0 in the appropriate column for each structure. Presence of the problem is indicated by 1, and absence of the problem is designated by 0.

Table 3. Structure Inspection Results (Partial).

Structure	Category	Problem								Date Constructed
		1	2	3	4	5	6	7	8	
BASKET #1	3	0	0	0	0	0	0	1	1	1993
BEARTAIL #1	2	0	0	0	0	0	1	0	0	1994
BEARTAIL TRIB #1	3	0	0	0	0	0	0	0	0	1994
BLACK #1	3	0	0	0	1	1	0	1	1	1991
BLACK #2	3	0	0	0	0	0	0	1	1	1994
CAMPBELL #1	3	0	0	0	0	0	0	0	1	1992
CANEY #1	3	1	0	0	0	0	0	0	1	1992
CANEY #2	2	1	1	0	1	0	0	1	1	1988

Displacement of the riprap from the weir, Problem 1, can be attributed to hydraulic forces caused by the impacting nappe. Previous physical model studies at CSU by Abt et al. (Abt et al. 1990, Abt et al. 1992) have demonstrated that many of the earlier structures were constructed with a smaller sized riprap than is required for stability at the conditions encountered by the structures. Three methods have been utilized in the later structures to achieve better riprap stability: (1) larger stone has been sized by the design curves developed by WES and CSU, (2) the riprap immediately downstream of the weir has been grouted, and (3) a modification has been implemented that provides a flat impact zone for the nappe as opposed to the sloping impact zone of the earlier structures. The 1993 field evaluation indicates that all three of the methods provide improved stability; however, more experience with these methods is required to develop confidence with either of the new modifications. Forty-one percent (41 %) of the structure evaluated in 1993 and 37% of the structures evaluated in 1995 have displacement of the riprap at the weir; however, Table 4 provides a history of the structure construction dates and the percentage of structures affected by Problem 1.

Table 4. Problem One History.

Construction Date	1986	1987	1988	1989	1990	1991	1992	1993	1994
% Affected	100%	75%	100%	100%	80%	25%	38%	4%	0%

These data indicate that the model testing by CSU and WES in 1991 and 1992 have been applied in the field with positive results, resulting in a decrease in structure instability.

Failure of the channel bank upstream or downstream of the structure, Problem Two, was observed in 37% of the structures evaluated in 1993, and only 23% of the structures inspected in 1995. Cause for this bank failure can be attributed to four reasons: (1) turbulence caused by transition from the graded, riprapped bank to the natural bank; (2) improper alignment of the flow into the structure; (3) gullying along either the upstream or downstream edge of the riprap caused by overbank drainage; and (4) gravity failure of the downstream bank due to channel incision. In the last two or three years, the Vicksburg District has extended a riprap toe upstream and downstream of the structures, and this modification has reduced the incidence of channel bank failure upstream or downstream of the structure. Addition of toe riprap to many of the existing structures has been recommended as a restoration measure.

The decrease in the percentage of structures with upstream or downstream channel banks failing can be attributed to improvements by the Vicksburg District, stabilization by colonizing vegetation, and to the large number of new structures that have been completed since the 1992 construction season. Most of the newer structures have a riprap longitudinal toe upstream and downstream of the basin.

Overbank drainage, Problem 3, is a problem of which the Vicksburg District is well aware; however, the problem is difficult to solve. Relatively minor slope changes, field modification of the intended site drainage plan, cattle denuding the slopes, or inconsistent material compaction can contribute to major overbank drainage erosion and piping. Poor overbank drainage control contributed to instability at 24% of the structures evaluated in 1993, and has decreased to 17% in 1995. In addition to rill and gully development, which can be visually assessed, many sites had evidence of piping of overbank drainage beneath the basin riprap. At these sites, the riprap blanket is depressed by as much as 1 to 2 feet, and the extent of the erosion and damage to the riprap filter is unknown. At this time, recommendations for restoration of problems caused by overbank drainage include installation of a drop pipe structure, filling and regrading of rill and gullies, and placement of a gravel filter at the upper rock rim. Emphasis placed on overbank gullies since the 1993 inspection seems to have improved site drainage conditions.

Launching of stone in the upstream or downstream apron, Problem Five, was observed at 24% of the structures evaluated in 1993, and has increased to 43% of the structures in 1995. In general, this is caused by local scour problems upstream or by incision of the downstream channel. Recommendations include replacement of the apron stone and placement of a heavy stone toe upstream and downstream of the structure. Severe head cutting into the basin was observed to occur in 17% of the structures evaluated in 1993, and in 37% of the structures in 1995. In these cases of severe head cutting from downstream incision, some form of additional downstream grade control may be required. The structures appear to be functioning well, with the riprap apron launching to protect the basin. This is an alarming increase in stream instability, and portends continued unstable reaches in the DEC channels.

Woody vegetation or heavy woody debris impairing conveyance at the structure, Problem Six, was observed at 19% of the sites evaluated in 1993 and in 11 % in the 1995 inspection. Debris should be removed if the hydraulics of the structures are being compromised. The effect of woody vegetation is not completely known. Inmost cases, the woody vegetation grows on sediment deposited within the basin and is washed away periodically resulting in young trees perhaps no more than three years old. However, several structures have older trees growing throughout the basin and at the toe of the approach apron. Consideration should be given to the effect on conveyance and on restricting the effective weir width caused by the older trees. Consideration should also be given to ecological factors in planning any type of vegetation removal program. Many of the structures evaluated, about half, were under construction or had not been begun at the time of the 1993 inspection. Newer structures are less likely to have significant woody vegetation.

The 1995 inspection has provided the resources to survey and inspect 1000 feet downstream of the structure and 1500 feet upstream of the structure, the 1993 inspection was confined to the immediate structure site. Problem Seven was identified at 69% of the structures inspected, which amplifies the significant increase in downstream apron launching (Problem 4) and head cutting migrating into the basin (Problem 5) that was previously discussed.

The thalweg elevation upstream of the structure was below the weir crest for more than 500 feet upstream, Problem Eight, in 78% of the structures. This can be interpreted as insufficient sediment is available, or the hydraulic conditions of the upstream channel and structure are inappropriate to cause storage of the sediment. In most streams within the DEC, sediment supply is plentiful. For the 65 structures inspected, the average distance upstream that the thalweg is below the weir crest is 899 feet, which implies that approximately 11 miles of channel sediment storage is not occurring because the low-flow channel does not allow sediment aggradation. Investigation should be made to determine modifications at the weir crest, or upstream of the structure, that would restrict low-to-moderate discharges to a degree that promotes deposition.

SUMMARY AND RECOMMENDATIONS

A summary of the results of this investigation is presented, followed by recommendations. Because of the complexity of the ongoing projects within the DEC watersheds, it is recognized that the recommendations, and to a certain extent the interpretation of the results, require discussion and feedback from Vicksburg District and Waterways Experimentation Station personnel before being finalized. The Principal Investigators welcome the opportunity to discuss these findings.

SUMMARY

All of the Category 1 structures designated in the 1993 investigation have been rehabilitated. The total number of structures in Category 1 or Category 2, those that have important problems, has decreased from 1993 to 1995. In the 1995 investigation, 77% of the structures were found to have no significant problems. An improvement in design, resulting in better structure performance and durability has been initiated in the 1991 through 1994 construction, which eliminates the displacement of riprap from the face of the weir. Bank stability immediately upstream and downstream of the structures has also improved, reducing bank instability from 37% of structures inspected in 1993 to 23% of structures inspected in 1995. Erosion problems resulting from overbank drainage at the sites was found to be decreased from 24% in 1993 to 17% in 1995. The likelihood of woody vegetation impairing structure conveyance was found to be reduced by 8%. These findings represent documented improvements in structure durability and performance.

Although neither situation directly impacts structure integrity nor durability, two general situations were identified: (1) The amount of active incision found downstream of the structures was found in 1995 to be increasing over 1993 findings, with active incision found within 1500 feet downstream of 69% of the structures investigated. (2) The thalweg elevation upstream of the structure was below the weir crest for a distance of more than 500 feet in 78% of the structures, indicating that the existing structures are not maximizing sediment storage and grade control.

RECOMMENDATIONS

Based on the evaluation of the structures, the following recommendations are presented:

1. Restore Category 1 structures as presented in Table 3 in accordance with the recommendations given on each structure evaluation form as soon as possible.

2. Develop a regular maintenance program to restore Category 2 structures, resolving the problems identified in the 1995 field evaluation.

3. Establish a regular annual field evaluation program of all structures: Category 1, Category, 2, Category 3, and new structures.

4. Develop a policy for maintenance of woody vegetation within the structures.

5. Develop a program for investigating and testing of erosion control methods for the structure construction site and for overbank drainage following construction.

6. Investigate modifications to existing structures or other channel modifications that would enhance sediment storage upstream of grade control structures.

7. Continue to design and construct grade control in the DEC streams to combat the increase in incision noted downstream of the structures.

8. Because of the durability and function of the major low-drop and high-drop structures, consider developing less costly structures that could implemented upstream of existing structures that would increase the extent of upstream control.

GRADE CONTROL STRUCTURE FIELD EVALUATION

COLORADO STATE UNIVERSITY

Stream & Structure No.	Basket No.1		(No.61)
Inspection Date	September 1995	Watson & Abt	09/12/95
Type of Structure	ARS-Type Low Drop Concrete cap, CSU type, Grouted Apron		Constructed 1993

OBSERVATIONS:

Upstream Apron	Grouted, stable
Upstream Side Slopes	Vegetation on toe and ½way up slope Stable - Rock settlement along headwall of weir
Downstream Side Slopes	Basin slopes stable, light vegetation
Weir	Good condition, vegetation (viney) at left toe
Baffle	Good condition, bottom plate 2' above water surface Collecting woody debris
Immediately Below Weir	Vertical drop, open pool
Plunge Pool	Open
Downstream Apron	Stable, good condition
Debris	Moderate woody debris throughout basin
Vegetation	Light throughout basin
Overbank Drainage	Overbank Drainage well vegetated. No gullying observed
Other	Downstream longitudinal toe has considerable launching Several headcuts moving upstream into basin

RECOMMENDATIONS:
Roll lA, photos 16-23
Category 3
No flow in creek
Pooling upstream, sand deposits observed
Generally good condition, headcuts will test integrity

CONSTRUCTION ISSUES <OPTIONAL>

<We consider this "frosting." If we have enough time we will cull some of our aggregate experience for short sections on the following topics.>

PROJECT PLANNING

<Includes staging, sequencing, and multi-phasing projects. Also coordination between design and construction. >

PERMITTING

<We have at least two examples of state-based guides for this topic. I have the one from Montana, while Marty has given me a reference to the Washington guide.>

ENVIRONMENTAL ISSUES

<This section, if we have time, will focus on issues such as habitat creation/destruction, fish passage, water chemistry and quality, large woody debris, vegetation, and others.>

SITE PREPARATION & RESTORATION

<The underlying emphasis of this section is public relations. Namely, good public relations. Most of the grade control work is done on private ground. Therefore, taking care, getting in, doing the work, and cleaning up when done, is an important human issue. There is also a fairly direct tie-in to Environmental Issues and Permitting.>

ANNOTATED BIBLIOGRAPHY

Different terminology and symbols are used by different authors. For this ANNOTATED BIBLIOGRAPHY, structure geometry and hydraulic performance are described in terms that are in general agreement with USACE practice. The following Federal agencies, laboratories, and organizations are commonly referred to by acronym:

Table 5. Organizational acronyms.

Acronym	Organization
USDA	US Department of Agriculture
ARS	Agricultural Research Service, US Department of Agriculture
NRCS (SCS)	Natural Resource Conservation Service (formerly Soil Conservation Service), US Department of Agriculture
USBR	Bureau of Reclamation, US Department of the Interior
USGS	US Geological Survey
USACE	Office, Chief of Engineers, US Army Corps of Engineers
WES	US Army Engineer Waterways Experiment Station
ASCE	American Society of Civil Engineers

Two recent reviews of grade control structure literature and design practice are included in this ANNOTATED BIBLIOGRAPHY. These reviews are particularly notable:

i. Nakato (1989) extends citations into foreign grade control structure design practice.

ii. McLaughlin (1986) includes various types of rock structures.

Other terms have been used in place of "grade control structure. " Department of Agriculture studies (Woolhiser and Lenz 1965) mention "gully control" structures, for example. USACE practice (USACE 1970) has been to consider "sediment control" structures in three categories:

i. Stabilizers

ii. Drop structures

iii. Debris basins and check dams.

Constraints hydraulic structure design includes:

i. Economics. Basin wide benefits, for example, flood control or bank protection, are economic factors that initially determine the need for upgrading a stream. A great latitude exists as far as materials and construction methods for grade control structures are concerned. For smaller streams, significant savings are obtained by selecting a structure that is locally inexpensive, provided the structure is durable and of low maintenance and meets other design constraints.

ii. Environmental. The relationships between a watershed and the concurrent range of dependent environmental factors are affected by the design of the grade control structure. Significant differences in environmental impact can be obtained by altering stage-discharge or sediment retention characteristics, as examples, in the structure selection and design process.

Design parameters, as listed by Goitom and Zeller (1989) for soil-cement structures, include the following topics:

Typical Hydrograph	**Land Cost and Use**
Velocity Range	**Aesthetics**
Discharge Range	**Safety**
Flow Duration	**Construction Material**
Low Flow Conditions	**Construction Costs**
Scour Potential	**Maintenance**
Aggradation Potential	

The sedimentation evaluation (scour and aggradation, above) determines the spacing between structures along a stream and the drop height for each structure. Bank and bed protection methods near a particular type of grade control structure are usually included in a documented description of the structure. However, the overall stream channel sedimentation evaluation is only rarely included and is not directly addressed in this ANNOTATED BIBLIOGRAPHY. The Goitom and Zeller document, which does include channel sedimentation, outlines the following study topics:

i. Determine the dominant discharge.

ii. Determine the hydraulic parameters for the dominant discharge.

iii. Determine the characteristics of the streambed sediments.

iv. Select appropriate sediment-transport relationship.

v. Estimate the long-term sediment supply at dominant discharge that is expected to be delivered to the reach under future watershed conditions.

vi. Compute the sediment-transport capacity of the study reach at dominant discharge.

vii. If supply is equal to capacity, then adjust the channel slope to obtain an equilibrium slope for which supply is equal to capacity at dominant discharge or armoring controls.

viii. Using the equilibrium slope, compute the spacing of the grade control structures (based on equilibrium slope, actual channel slope, and drop height).

ix. Using an appropriate formula, compute the expected streambed scour downstream of the grade control structure.

x. Bury invert of the grade control below the expected scour depth.

A structure may be designed either uniquely for grade control or to provide grade control in conjunction with an alternate primary function. Most citations herein deal with a single type of structure as noted in the annotations. Citations having design guidance applications for larger agencies usually discuss several types of structures. For example, a survey of structures presently used in the Denver, CO, area (McLaughlin Water Engineers, Ltd., 1986) includes the following types:

Type of Structure	Number of Cases
Baffled apron drops	5
Vertical drop with loose riprap basin	5
Vertical drop with hard basin	14
Sloping rock drops	12
Sloping grouted rock	7
Sloping concrete drops and other hard basins	4
Low-flow erosion checks and control measures	5

USACE guidance (1970) includes a discussion of three types of sediment control situations that enter into the selection of type of structure:

i. Stabilizers are designed to limit channel degradation. Two structural designs, a rock stabilizer and a sheet piling stabilizer, are provided.

ii. Drop structures are designed to reduce channel slopes to effect non-scouring velocities. Details and design charts for a typical drop structure are included.

iii. Debris basins and check dams are built in the headwaters of flood control channels having severe upstream erosion problems in order to trap large bed-load debris before it enters main channels. A typical design, including both a spillway and a pool drain, of a debris basin is included.

The Bureau of Reclamation design guidance for hydraulic structures (1978a, 1978b, 1987) provides details for a broad range of structures. Most of these structures are viable options for grade control applications in particular field situations.

<Redo this brief introduction>

This ANNOTATED BIBLIOGRAPHY provides information sources applicable to matching design options to diverse functional needs. The definition of function in terms of sedimentation issues such as degradation and aggradation is not adequately addressed in the citations. Since these issues directly impact on structure height and spacing, further research, for the benefit of providing comprehensive design guidance to the overall grade control problem, appears necessary.

A

1. J.H. Ables Jr., M.B. Boyd, 1969. "Low-Water Weirs on Boeuf and Tensas Rivers, Bayou Macon, and Big and Colewa Creeks, Arkansas and Louisiana; Hydraulic Model Investigation." Technical Report H-69-13, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

A 1:20-scale model reproducing one-half of the channel section through a low-water weir was used to test eight plans of channel improvement. The tests were conducted with a drop height of 5.5 ft, and discharges up to 10,000 ft^3/s were observed. Flow conditions and slope protection downstream from the weir to control the flanking problem were found to be satisfactory with channel improvement plans 6 and 8. Riprap with an average weight of 33 lb. was used in slope protection in both plans. A model headwater-tailwater rating curve was obtained. A second 1:20-scale section model was constructed in a 2.5-ft-wide glass flume for more generalized riprap tests in the vicinity of the structure. Several drop structure plans were observed to develop an effective, economical drop structure. Of primary concern were development of some of the dimensions of various elements of the structure and determination of riprap requirements in the vicinity of the structure. Limiting tailwater curves for 220- and 325-lb. riprap are furnished as a guide in riprap selection at drop structures with drop heights of from 5 to 10 ft. Suggestions as to the use of these curves also are included. A basin length of 20 ft is considered to be adequate at projects where the jet will ride through at unit discharges exceeding about 10 ft^3/s. A riprap plan which provides additional protection to the structure is presented in Appendix A.

2. J.H. Ables Jr., G.A. Pickering, 1975. "Flood Control Project on Lytle and Warm Creeks and Santa Ana River, California; Hydraulic Model Investigation." Technical Report H-75-7, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

The Lytle and Warm Creeks and Santa Ma River project will provide another important unit under the general comprehensive plan for flood control in San Bernardino County, California. The proposed plan for containing the flood flows will consist of raising levees; excavating streambeds; and constructing grade control structures, energy dissipaters, bridges, and several thousand feet of high-velocity concrete channels. The investigation was conducted on a 1:60-scale model that reproduced approximately 10,000 ft of the Santa Ma River, 600 ft of East Twin Creek, and 5,300 ft of Warm Creek. The existing and proposed bridges, concrete channels, and natural streambed channels with revetted slopes were also reproduced in the model. Tests were concerned with flow conditions; water-surface elevations; riprap stability; and sediment transport at the grade control structures, bridges, confluences, and energy dissipaters.

3. J.H. Ables Jr., 1976. "Divide Cut Drainage Structures, Tennessee-Tombigbee Waterway, Mississippi and Alabama; Hydraulic Model Investigation." Technical Report H-76-18, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Standardized designs for drainage structures in the Divide Cut Section of the Tennessee-Tombigbee Waterway were developed in model tests conducted on the USBR type VI impact basin at a scale of 1:4, on the minor drainage chutes and energy dissipaters at a scale of 1:10, and on the major drainage structures at a scale of 1:25. Test results indicate that the type VI impact basin performs satisfactorily below rectangular channels for all discharges tested, and critical dimensions are tabulated for discharges expected at drainage structures where the type VI basin will be installed. Generalized information was developed to permit satisfactory design of minor drainage chutes and energy dissipaters emptying into the canal. A satisfactory baffled chute spillway was developed for the largest of the drainage structures. Model test results will permit design of the other three major structures based on a unit discharge of 60 ft^3/s common to the five structures for 100-year frequency flows.

4. S. Adachi, T. Nakato, 1969. "Changes of Top-Set-Bed in a Silted Reservoir." Proceedings of the 13th Congress of the IAHR, Kyoto, Japan.

It is the purpose of this paper to show that changes of the river bed due to the top-set-bed may be characterized simply by a diffusion equation. The diffusion coefficient is also evaluated for a natural river, the Tenryu River.

5. J. Aguirre, J.R. Achinte, H.J. Jegat, 1980. "Estudio Experimental de la Socavacion Local en Una Estructura de Caida de Seccion Trapecial." Proceedings of the 9th Congresso Latinoamericano de Hidraulica, Vol. 1, Merida, Venezuela, June 30-July 4, 1980 (in Spanish). pp. 447-456.

It is the intent of this work to study the behavior of a drop structure of a trapezoidal section on non-cohesive homogeneous soil. The study was performed on a scaled model. Based on the experimental results, equations are proposed to determine the maximum scour depth and the distance at which it occurs.

6. K. Ashida, T. Takahashi, T. Mizuyama, 1975. "Some Discussions on Hydraulic Design of Waterway Stabilization Structures." New SABO, No. 97, 1975 (in Japanese). pp. 9-16.

Many drop structures and permanent bank protections are installed in a waterway to prevent lowering and enlargement of the channel cross-section. But the design rules or such structures are mainly based on experience and it is not rare to suffer from unexpected phenomena. This paper discusses the hydraulic function and design rules of waterway stabilization structures based on the results of sediment hydraulics and the hydraulics of channel controls.

7. P. Ackers, W.R. White, J.A. Perkins, A.J.M. Harrison, 1978. Weirs and Flumes for Flow Measurement. Wiley, New York.

This book covers both theoretical and practical aspects of water measurement using gaging structures. Its scope covers small discharges in laboratories and processing plants, as well as those larger discharges in rivers that remain amenable to measurement by this technique. The conditions under which weirs and flumes are likely to provide an appropriate method and standard of accuracy are described, and the criteria for selecting a suitable structure are explained. (193 references)

8. R. Agostini, A. Bizzarri, M. Masetti, and A. Papetti, 1988. "Flexible Gabion and Reno Mattress Structures in River and Stream Training Works; Volume 1: Weirs." Maccaferri Gabions, Inc., Williamsport, MD.

Contents: Chapter I: Training and Hydraulic Protection Structures; Chapter II: Gabion Weirs; Chapter III: Design Criteria for Vertical and Stepped Weirs: Construction Details; Chapter IV: Design Criteria for Sloped Weirs: Construction Details; Chapter V: Examples of Calculations; Chapter VI: Examples of Completed Works. (66 references)

B

9. F.W. Blaisdell, C.A. Donnelly, 1951. "Capacity of Box Inlet Drop Spillways under Free and Submerged Flow Conditions." Technical Paper No. 7, Series B, St. Anthony Falls Hydraulic Laboratory, University of Minnesota, Minneapolis, MN.

This paper presents methods for determining free flow capacity; tests and results; illustrated description of test apparatus; effects of position of dike, approach channel width, and shape of inlet; and correction factors.

10. F.W. Blaisdell, C.A. Donnelly, 1966. "Hydraulic Design of the Box-Inlet Drop Spillway." Agriculture Handbook No. 301, US Department of Agriculture, Agricultural Research Service, Washington, DC. Also published in 1951 as Technical Paper No. 8, Series B, St. Anthony Falls Hydraulic Laboratory, University of Minnesota, Minneapolis, MN.

The study resulted in the development of a generalized method for determining the free flow capacity of box inlet drop spillways. The procedure is outlined in this report. No practical generalized method for the determination of the submerged flow capacity was found; the submerged flow capacity is determined by a process of interpolation utilizing submergence curves obtained for a wide range of pertinent variables for this purpose.

11. F.W. Blaisdell, K.M. Hayward, C.L. Anderson, 1982. "Model-Prototype Scour at Yocona Drop Structure." Proceedings of the ASCE Conference 'Applying Research to Hydraulic Practice'. Jackson, MS. August 17-20, 1982. Peter E. Smith, ed.. pp. 1-9.

The scour measured at the exit of a straight drop spillway stilling basin model and the scour measured at its prototype are compared. There are three major parts to the paper: (1) features of the stilling basin design and its performance are presented; (2) the Yocona River prototype structure is described; and (3) a comparison of the model and prototype scour patterns concludes the presentation. (2 references)

12. E. Bloom, C.A. DeGraff, 1980. "Corrugated Aluminum Drop Structure for Erosion Control." ASAE paper 80-2046. 15 pp. 1980.

Economical low over-fall grade and erosion control structures are fabricated of standard corrugated aluminum structural plates and set in a reinforced concrete base. The standard sizes developed and the limitations of their use are presented.

13. N.E. Bormann, 1988. "Equilibrium Local Scour Depth Downstream of Grade Control Structures." Ph.D. Dissertation, Colorado State University, Fort Collins, CO.

A theoretical analysis of the diffusion and path of two-dimensional jets is combined with particle stability analysis so that local scour from a variety of conditions can be described using two parameters. The geometry of the flow and structure is described by the impingement angle at the boundary. Separate empirical relationships are developed to estimate impingement angle for free jets and submerged jets. Local scour dynamics are described by a scour stability parameter which is a ratio of the force of the diffused jet velocity causing scour to the gravitational force resisting scour.

To confirm the theoretical analysis, an extensive, large-scale experimental program was completed using unit discharge rates of up to 25 $ft^3/s/ft$. The data collected are unique in three respects: (1) provide near-prototype-scale experimental local scour data; (2) test various face slopes of structures; and (3) test partially submerged flow conditions. The data collected are combined with four data sets available in the literature to represent a variety of flow and geometric conditions.

A sensitivity analysis of the predictive equations for local scour depth illustrates the disproportionately large effect small errors in values of the variables describing local scour have on predictions. This sensitivity is reflected in the large experimental scatter present in all local scour data.

14. M.G. Bos, ed., 1976. "Discharge Measurement Structures." Publication 20, International Institute for Land Reclamation and Improvement, Wageningen, The Netherlands.

This report presents instructions, standards, and procedures for the selection, design, and use of structures which measure or regulate the flow rate in open channels. The topics discussed cover lists of principal symbols, auxiliary weirs, sharp-crested weirs, short-crested weirs, flumes, orifices, miscellaneous structures, basic equations of motion in fluid mechanics, the overall accuracy of the measurement of flow, side weirs and oblique weirs, and suitable stilling basins. (173 references)

15. A.M. Brate, 1989. "Wooden grade stabilization structures." ASAE paper 892620. 9 pp.

This paper describes the use of treated lumber to construct grade stabilization structures in solving gully erosion problems. Design, economic, and construction considerations are presented.

16. J.A. Brevard, 1971. "Criteria for the Hydraulic Design of Impact Basins Associated with Full Flow in Pipe Conduits" <>(see complete citation under US Department of Agriculture).

17. J.A. Brevard, 1975. "Hydraulic Design of Riprap Structure for Channel Gradient Control." Proceedings of the 68th Annual Meeting of the American Society of Agricultural Engineers. University of California, Davis, CA. June 22-25, 1975. Paper No. 75-2022.

Some channels can be stabilized economically using a riprap gradient control structure. The riprap structure consists of a prismatic channel with a transition at each end. For the design discharge, the riprap structure is designed to maintain a constant specific energy head through the structure. (4 references)

18. <u>Design of Small Canal Structures.</u> US Bureau of Reclamation. 1978a. Denver, CO.

Contents: Chapter I: General Requirements and Design Considerations (9 refs); Chapter II: Conveyance Structures (14 refs); Chapter III: Regulating Structures (3 refs); Chapter IV: Protective Structures (11 refs); Chapter V: Water Measurement Structures (5 refs); Chapter VI: Energy Dissipaters (10 refs); Chapter VII: Transitions and Erosion Protection (1 ref); Chapter VIII: Pipe and Pipe Appurtenances (12 refs); Chapter IX: Safety (3 refs); Appendix A: Glossary of Terms; Appendix B: Conversion Factors; Appendix C: Computer Program.

19. <u>Hydraulic Design of Stilling Basins and Energy Dissipaters.</u> Engineering Monograph No. 25. US Bureau of Reclamation. 1978b. Denver, CO.

Contents: Section 1: General Investigation of the Hydraulic Jump on Horizontal Aprons (Basin I); Section 2: Stilling Basin for High Dam and Earth Dam Spillways and Large Canal Structures (Basin II); Section 3: Short Stilling Basin for Canal Structures, Small Outlet Works, and Small Spillways (Basin III); Section 4: Stilling Basin Design and Wave Suppressors for Canal Structures, Outlet Works and Diversion Dam (Basin IV); Section 5: Stilling Basin with Sloping Apron (Basin V); Section 6: Stilling Basin for Pipe or Open Channel Outlets (Basin VI); Section 7: Slotted and Solid Buckets for High, Medium, and Low Dam Spillways (Basin VII); Section 8: Hydraulic Design of Hollow-Jet Valve Stilling Basins (Basin VIII); Section 9: Baffled Apron for Canal or Spillway Drops (Basin IX); Section 10: Improved Tunnel Spillway Flip Buckets (Basin X); Section 11: Size of Riprap to be Used Downstream from Stilling Basins. (75 references) Contents: Chapter 1: Plan Formulation; Chapter 2: Ecological and Environmental Considerations (31 refs); Chapter 3: Flood Hydrology Studies (9 refs); Chapter 4: Selection of Type of Dam; Chapter 5: Foundations and Construction Materials (28 refs); Chapter 6: Earthfill Dams (72 refs); Chapter 7: Rockfill Dams (47 refs); Chapter 8: Concrete Gravity Dams (14 refs); Chapter 9: Spillways (31 refs); Chapter 10: Outlet Works *(15* refs); Chapter 11: Diversion During Construction (1 ref); Chapter 12: Operation and Maintenance (2 refs); Chapter 13: Dam Safety (21 refs); Appendix A: Reservoir Sedimentation (41 refs); Appendix B: Hydraulic Computations (11 refs); Appendix C: Structural Design Data (13 refs); Appendix D: Soil Mechanics Nomenclature; Appendix E: Construction of Embankments (8 refs); Appendix F: Concrete in Construction (6 refs); Appendix 0: Sample Specifications (4 refs); Appendix H: Typical Checklist of Dams and Structures for On-site Inspection; Appendix I: Conversion Factors.

C

20. S. Colyer, 1987. "Fishing the Four-Lane." Civil Engineering. Vol. 47, No. 8. pp. 50-51.

Recent changes in one Montana canyon have shown that highways and freshwater fish can coexist. Twenty years ago, the Montana Department of Highways began plans to construct one of Interstate 15's final stretches. To preserve the habitat of the native trout and whitefish, engineers took a number of measures to maintain existing fish cover by building shore-anchored structures and planting them with new grasses and shrubs. Artificial rock drop structures, sunken cover, and log check dams were also used to create new fish holding areas. Erosion control structures were also built.

21. C.M. Cooper, S.S. Knight, 1987. "Fisheries in Man-Made Pools Below Grade-Control Structures and in Naturally Occurring Scour Holes of Unstable Streams." Journal of Soil and Water Conservation. Vol. 42, No. 5. pp. 370-373.

As part of the ecological research on high-gradient streams in the Yazoo River Basin of Mississippi, four man-made pools below grade control (low-drop) structures and four naturally occurring scour hole pools were sampled for fish composition by the rotenone method. Tillatoba and Long Creeks were chosen because of the presence of grade control structures used as structural management practices for control of channel erosion from head cutting and because the region has been included in a comprehensive land treatment and channel stability project. Differences in the fisheries characteristics of the two pool habitats were expected because of differences in their relative stability, bottom substrate, and pool life expectancy. Natural scour holes yielded 0.06 kg/cu m of fish from 39 species; 0.018 kg/cu m were considered harvestable. Twenty-nine species in man-made pools yielded 0.06 kg/m^3 with 0.025 kg/m^3 being harvestable. Length frequency distribution indicated that there was better growth of many species and more stable reproducing populations of forage fish in man-made pools, although they yielded somewhat fewer species. Drop structure pools have several advantages over most natural scour holes in their fisheries characteristics as well as providing protection from stream channel degradation. (5 references)

22. M.L. Corry, P.L. Thompson, F.J. Watts, J.S. Jones, D.L. Richards, 1983. "Hydraulic Design of Energy Dissipaters for Culverts and Channels." Report No. FHWA/EPD-86/110, HEC-14, Federal Highway Administration, Washington, DC.

The manual provides information for designing energy dissipaters for culvert outlets and for drops in open channels. Design procedures and charts are provided for selecting and sizing impact basins, hydraulic jump basins, riprap basins, and drop structures. The basic hydraulic design concepts for energy dissipation are presented, and application of these concepts to the design of various types of basins is illustrated through the use of example problems.

D

23. D.G. DeCoursey, 1981. "Stream Channel Stability; Comprehensive Report: Project Objectives 1 Through 5." prepared for US Army Corps of Engineers, Vicksburg District, Vicksburg, MS, under Section 32 Program, Work Unit 7, USDA-ARS National Sedimentation Laboratory, Oxford, MS.

This volume was written to describe how to carry out an analysis of a channel stability problem. Detailed reports of the various research projects associated with channel stability problems in the Yazoo River Basin are presented as appendices. The overall research program had five major objectives: (1) determining the influence of grade control structures on channel stability; (2) monitoring the performance of selected channel stabilization methods; (3) evaluating the effects of geology, geomorphology, soils, land use, and climate on runoff and sediment production from major source areas; (4) estimating the water and sediment production from a large, mixed-land-use watershed and the integrated effects on channel stability; and (5) evaluating the relation between valley stratigraphy and channel morphology and their combined effects on channel stability. (125 references)

24. C.W. Denzel, C.N. Strauser, 1982. "Kaskaskia River Grade Control Structure." Proceedings of the 1982 International Symposium on Urban Hydrology. Hydraulics and Sediment Control. Lexington, KY. July 27-29, 1982. University of Kentucky, Water Resources Research Institute. pp. 135-139.

The Kaskaskia River control structure is located near the US Highway 460 bridge at Fayetteville in St. Clair County, Illinois. Since the completion of the navigation channel in 1972 from Fayetteville to the confluence with the Mississippi River, some 36 canal miles, head cutting and channel widening have occurred upstream of Fayetteville. Downstream of Fayetteville, about 2.5 million cubic yards of sediment have been deposited in the upper 6 miles of the navigation canal. A grade control structure is required so that upstream head cutting and downstream sedimentation are minimized The design of the structure was based on theoretical computations and verified by model test at the US Army Engineer Waterways Experiment Station. It is concluded that a structure is needed in order to maintain the upstream water surface profile so as not to disturb the state of dynamic equilibrium which presently exists in the channel, both upstream and downstream of Fayetteville. (1 reference)

25. C.A. Donnelly, F.W. Blaisdell, 1954. "Straight Drop Spillway Stilling Basin." Technical Paper No. 15, Series B, St. Anthony Falls Hydraulic Laboratory, University of Minnesota, Minneapolis, MN. Also published in Journal of the Hydraulics Division, ASCE, Vol. 91, No. HY3. pp. 101-131.

This paper describes the development of generalized design rules for a stilling basin for use with the straight drop spillway. This stilling basin design was developed because experience in the field had shown that there was no satisfactory stilling basin for the straight drop spillway. Limited field experience indicates that the design adequately protects the downstream channel from scour.

Water failing over the spillway crest falls onto a flat apron. The nappe is broken up by floor blocks, which also prevent damaging scour of the downstream channel banks. An end sill prevents scour of the downstream channel bed. Flaring wing walls, triangular in elevation, prevent erosion of the dam fill. For proper operation of the stilling basin, the contraction of the flow at the ends of the spillway opening must be partially suppressed.

An important finding is that the stilling basin length computed for the minimum tailwater level required for good performance may be inadequate at higher tailwater levels. Dangerous scour of the downstream channel may occur if the nappe is supported sufficiently by high tailwater so that it lands beyond the end of the stilling basin. A method of computing the stilling basin length for all tailwater levels is presented. (9 references)

E

26. A.M. El Khashab, 1986. "Form Drag Resistance of Two-Dimensional Stepped Steep Open Channels." Canadian Journal of Civil Engineering, Vol. 13, No. 5. pp. 523-527.

Flow in rough steep open channels is found mostly in mountain streams and in flow overtopping protected weirs. In both cases, the energy of the flowing stream may be dissipated by artificial means so that the flowing water does not result in serious damage due to scour or erosion downstream of the main slope. One way of achieving this purpose is to lead the flow over a series of steps. (12 references)

27. A.M. El Khashab, O. Helweg, A. Alajaji, 1987. "Theoretical Flow Model for Drop Structures." Proceedings of the ASCE National Conference on Hydraulic Engineering. Williamsburg, VA. August 3-7, 1987. Robert M. Ragan, ed.. pp. 1112-1117.

A theoretical model based on the momentum equation is reviewed for a trapezoidal drop structure operating in a trapezoidal channel. Preliminary qualitative results indicate agreement between the theoretical and observed data. (4 references)

F

28. J. Farhoudi, K.V.H. Smith, 1985. "Local Scour Profiles Down-stream of Hydraulic Jump." Journal of Hydraulic Research, IAHR, Vol. 23, No. 4. pp. 343-358.

The development of local scour holes downstream of a hydraulic jump flow during the passage of time shows certain geometrical similarities which may be expressed by relevant parameters. The paper attempts to explain the similarity existing either in the process of scour or in the profiles that the scour holes follow downstream of hydraulic jump flow. The investigation was carried out using three geometrically similar models with geometrical scale progressing by a factor of two. (9 references)

29. W.R. Ferrell, W.R. Barr, 1963. "Criteria and Methods for Use of Check Dams in Stabilizing Channel Banks and Beds." Proceedings of the Federal Interagency Sedimentation Conference. Subcommittee on Sedimentation, ICWR. Jackson, MS. Miscellaneous Publication No. 970, Paper No. 44, US Department of Agriculture, Agricultural Research Service. pp. 376-386.

Through experience and experimentation a procedure has been developed that reduces the time required to prepare plans for a stabilization system. The construction of stabilization structures is organized on an assembly line basis. Once the structure is completed, certain repair work will be required, particularly in the area of the spillway. Experience has shown that annual inspections are needed after each storm season to assure correction of maintenance problems.

30. A.A. Fiuzad, 1987. "Head Loss in Submerged Drop Structures." Journal of the Hydraulics Division, ASCE. Vol. 113, No. 12. pp. 1559-1562.

This note describes a model study of a drop structure where, for the 100-year flood discharge, the height of drop was smaller than the depth of flow in the channel. The structure was to be used for grade control on a storm drainage channel. The purpose of the study was to find an economical stilling basin for the structure. (3 references)

31. B.P. Fletcher, J.L. Grace, Jr., 1973. "Cellular-Block-Lined Grade Control Structure; Hydraulic Model Investigation." Miscellaneous Paper H-73-7, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

A 1:4-scale physical model study was conducted to investigate flow characteristics and develop the geometry required for a grade control structure lined with cellular blocks to accomplish a 4-ft change in grade within a trapezoidal channel conveying a discharge of 432 ft^3/s.

32. A.W. Fogle, J.C. McBurnie, B.J. Barfeld, K.M. Robinson, 1993. "Modeling Free Jet Trajectory at an Overfall and Resulting Shear Stress Distribution in the Plunge Pool." Trans. Am. Soc. Agricultural Engineers. 36 (5):1309-1318.

A model of free jet trajectory at an over fall and the resulting shear distribution in the plunge pool has been developed. The model utilizes end depth relationships and simple particle physics to predict jet trajectory. Classical jet theory is coupled with empirical information to develop algorithms for predicting plunge pool shear distributions. The model performs well predicting the trajectories of well-ventilated jets but requires improvement when dealing with non-ventilated jets. The model performs satisfactorily predicting shear distributions below an over fall.

33. P. Forsythe, 1985. "Performance of a Grade Control Structure System During Extreme Floods." Proceedings of the 1985 Winter Meeting of the American Society of Agricultural Engineers. Chicago, IL. December 17-20, 1985, Paper 85-2622.

A series of five grade control structures installed in 1980 have been subjected to three major floods since construction. The hydraulic performance of the structures and stabilization of the channel have been excellent. The straight drop structures used were found to provide flow and sediment-transport retardation. (7 references)

34. E.P. Fortson, Jr., G.B. Fenwick, J.J. Franco, H.S. Austin, 1957. "Flood Control Project, Hoosic River, Adams, Massachusetts; Report Number 2: Model Investigation of Phase II of Improvement Works." Technical Memorandum 2-339/2, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Model investigations were conducted to verify hydraulic design of the improvement plan and to determine modifications required to provide the most economical and hydraulically sound design for high-velocity flow. A 1:20-scale model reproducing 6,800 ft of the main channel of Hoosic River and the lower 212 ft of Fiske Brook was used to check channel alignment, super elevation in bends, characteristics of weirs, stilling basin, drop structures, and treatment of intakes and outlets. Tests of the original design indicated flow conditions to be generally satisfactory. Revisions were made to improve flow conditions at the approach to a weir, at two diversion structures, and at Fiske Brook outlet. Special studies were made of minimum elevation required for a" bridges. Side wall heights necessary to prevent spillage were determined.

G

35. R.A. Gemma, R. Li, D.B. Simons, 1982. "Soil-Cement Grade Control Structures." Proceedings of the 1982 International Symposium on Urban Hydrology Hydraulics and Sediment Control. Lexington, KY. July 27-29, 1982. University of Kentucky, Water Resources Research Institute. pp. 125-129.

Grade control structures are used to stabilize stream channels where general degradation and head cutting threaten the stability of existing structures or otherwise create the potential for financial loss.

An alternative to a large reinforced concrete structure is a series of smaller grade control structures constructed of weaker, less expensive materials. Where sandy soils are present, grade control structures may be constructed of soil-cement. This approach to channel stabilization was recently taken in Pima County, Arizona, by Simons, Li and Associates, Inc. This paper presents the developed design methodology. (4 references)

36. J.F. George, G.A. Pickering, H.O. Turner, Jr., 1994. "General Design for Replacement of or Modifications to the Lower Santa Ma River Drop Structures, Orange County, California." Technical Report HL-94-4, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

The first part of this study was to investigate new vertical drop structures and sloping grouted stone structures to replace existing structures. Numerous designs were tested on a 1:25 scale. Modifications include a parabolic drop downstream from the crest, one and two rows of baffle blocks, and a sloping end sill. The second part of the study introduced modifications to the existing drop structures. The same scale and variations of the same modifications were studied for the existing drops as for the new designs.

The objectives of the overall program was to have the structures provide good energy dissipation within the basin and to minimize downstream scour over a range of discharges and tail waters. Unit discharges range from 125 to 250 $ft^3/s/ft$. Froude numbers for the jet entering the basin range from about 1.9 to 3. l. The testing during the first part of the study established a general design method for parabolic drop structures suitable for grade control projects that experience high unit discharge.

37. M.A. Gill, 1979. "Hydraulics of Rectangular Vertical Drop Structures." Journal of Hydraulic Research, IAHR, Vol. 17, No. 4. pp. 289-302.

Previous works of Moore, White, and Rand on the characteristics of the free over fall drop structures are briefly reviewed. A simple theory initially proposed by White is modified by introducing less drastic assumptions than the ones on which White's theory is based. The resulting theory is in closer agreement with experiment and the empirical equations of Rand than White's theory. Experimental results are also given. (9 references)

38. M.A. Gill, 1983a. "Diffusion Model for Aggrading Channels." Journal of Hydraulic Research, Vol. 21, No. 5. pp. 355-367.

The problem of aggradation in a rectangular channel of finite length due to an overloading of sediment supply is formulated in terms of a diffusion type of partial differential equation and suitable initial and boundary conditions are identified. Analytical solutions are presented for computations at large and snail values of tine.

39. M.A. Gill, 1983b. "Diffusion Model for Degrading Channels." Journal of Hydraulic Research, Vol. 21, No. 5. pp. 368-378.

Diffusion equation is used to describe degradation of channel beds below dams and in rectangular channels whose base level is abruptly lowered. Analytical solutions are presented for computations at large and small values of tine.

40. T.G. Goitom, M.E. Zeller, 1989. "Design Procedures for Soil-Cement Grade-Control Structures." Proceedings of the ASCE National Conference on Hydraulic Engineering. New Orleans, LA. August 14-18, 1989. Michael A. Ports, ed.. pp. 1053-1059.

Grade control structures are effective channel stabilization measures that may be used singly or as a part of a stabilization plan. Grade control structures can range in complexity from simple rock riprap to soil-cement drop structures to large concrete structures with baffled aprons and stilling basins. This paper presents a step-by-step design procedure for soil-cement grade control structures in sand bed channels, and the application of the procedure in practice is demonstrated by using a case study of a channelization project along San Diego Creek in the City of Irvine, CA. (3 references)

41. H.J. Goon, J.A. Brevard, 1976. "Hydraulic Design of Riprap Gradient Control Structures." Technical Release No. 59 (see complete entry under US Department of Agriculture).

42. J.L. Grace, Jr., P. Bhramayana, 1980. "Kaskaskia River Grade-Control Structure and Navigation Channel, Fayetteville, Illinois; Hydraulic Model Investigation." Technical Report HL-80-20, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

The structure proposed at the head of the Kaskaskia River navigation channel near Fayetteville was designed to maintain existing water surfaces without aggravating head cutting and upstream bank erosion. A 1:25-scale model investigation was conducted to evaluate the hydraulic performance of the structure including discharge characteristics of the weir, velocities of flow through the structure and in the downstream navigation channel, general flow patterns, and riprap needed for stability of the structure and for protection of the navigation channel. The proposed grade control structure is constructed of riprap and an additional 899 ft of bank protection will be provided immediately downstream. The total structure width is 600 ft and includes a trapezoidal weir with a base width of 120 ft. The original design structure performed satisfactorily with discharges ranging from 1,000 to 35,000 ft^3/s; however, the headwater was below desirable levels. Several modifications of the weir portion of the structure were developed to maintain existing headwaters and provide satisfactory hydraulic performance.

H

43. G.N. Hadish, M. Braster, R.A. Lohnes, C.P. Baumel, 1994. "Stream Stabilization in Western Iowa." Final Report submitted to the Iowa Department of Transpiration.

Since early in this century, streams in western Iowa have degraded 1.5 to 5 times their original depth. This vertical erosion is often accompanied by widening of two to four times the original width. The deepening and widening of the channels has jeopardized the structural safety of 25% of the 5,223 bridges in the area by undercutting footings or pile caps and removing soil beneath and adjacent to abutments. This report suggests methods for predicting the extent of degradation and locating grade control structures.

One counter measure to degradation is to design stream crossings with sufficient width and foundation depth to accommodate impending degradation. Two methods to predict the amount of vertical degradation are evaluated. One method is a geomorphic approach that identifies the stable reach of a degrading stream and graphically projects the longitudinal profile upstream to the degrading reach to estimate the future amount of down cutting. The second method is an analytical iterative process of balancing applied tractive force with erosion resistance. Both methods show promise of being useful but are applicable only to streams of uniform bed material.

Grade control structures are another counter measure to the threat to infrastructure from channel erosion. Historical evidence indicates that some structures were placed where they were not needed and others placed at a location where they were less effective than they could have been. With increasing costs, the selection and placing of grade control structures is presented. A flow chart is presented to help engineers assess the need for a grade control structure. Geologic conditions that influence the structures' foundation and channel side slope stability are described. Methods are suggested to estimate the reach of rivers that will be influenced by the grade control structure. Finally, the various components of the planning process re related in a second flow chart.

44. R.F. Hadley, 1963. "Characteristics of Sediment Deposits Above Channel Structures in Polacca Wash, Arizona." Proceedings of the Federal Interagency Sedimentation Conference. Subcommittee on Sedimentation, ICWR. Jackson, MS. Miscellaneous Publication No. 970, Paper No. 80, US Department of Agriculture, Agricultural Research Service. pp. 806-810.

Deposition caused by the construction of dams on Polacca and Wepo Washes since 1945 is estimated at 7,500 acre-ft. Longitudinal profiles were surveyed above each dam on Polacca Wash. The present channel gradients on the sediment deposits vary from 0.0005 ft/ft to 0.0037 ft/ft. The gradients of these channels prior to dam construction ranged from 0.0040 ft/ft to 0.0058 ft/ft.

The conclusion is that a change in stream regimen caused by a rising base level such as a dam will reduce the channel slope and cause aggradation upstream to a higher elevation than that of the channel control. The extent of this deposition may be affected by valley width, channel slope, particle size of available material, and influence of riparian vegetation. The relative importance of each of these factors has yet to be determined. (4 references)

45. W.H. Hager, N.V. Bretz, 1986. "Hydraulic Jumps at Positive and Negative Steps." Journal of Hydraulic Research, International Association of Hydraulic Research, Vol. 24, No. 4. pp. 237-253.

The hydraulic flow features associated with a hydraulic jump over positive and negative steps are investigated. Attention is paid to the length of the jump to use in the design of the stilling basin. In particular, the sequent depth ratio, the length characteristics, and the wave formation are analyzed by elementary means and confirmed by observations. A comparison of two energy dissipaters is presented. (15 references)

46. G.J. Hanson, R.A. Lohnes, F.W. Klaiber, 1986. "Gabions Used in Stream Grade-Stabilization Structures: A Case History." Hydraulics and Hydrology, Transportation Research Record 1073. Transportation Research Board, National Research Council. Washington, DC. pp. 35-42.

Grade stabilization structures have been effective in controlling the degrading of streams in western Iowa. However, the cost of reinforced concrete structures has risen to the point that less expensive materials need to be considered. In an effort to evaluate alternative materials, a gabion drop structure was designed, built, and monitored for 2 years after completion. A cost analysis that normalizes several variables is used to compare the gabion structure with concrete structures and indicates that the cost of building the gabion structure was about 20 percent of that of a comparable size concrete structure. It is concluded that this type of structure is an effective and economic alternative. (6 references)

47. M.D. Harvey, 1984. "A Geomorphic Evaluation of a Grade-Control Structure in a Meandering Channel." River Meandering: Proceedings of the ASCE Conference Rivers '83, New Orleans, LA. October 24-26, 1983. Charles M. Elliott, ed.. pp. 284-294.

The performance of a two-stage grade control structure located in Middle Fork, Tillatoba Creek, Mississippi, was evaluated 4 years after construction. The structure was designed to prevent the upstream migration of incision, and to provide flow control to rehabilitate eroded downstream reaches. Morphometric data for three channel conditions were analyzed: 1954 natural channel, 1975 pre-construction, and 1982 post-construction. The data show that flow control significantly reduced downstream channel erosion. Channel erosion is continuing upstream of the structure. The mean upstream channel slope (0.0014) is twice the design slope (0.0007) because of channel shortening during construction and a subsequent cutoff (2,970 ft (905 m)). Backwater conditions during bank-frill discharge (25-year flow) cause bank saturation and ultimately failure. Increased water- surface slope during recessional flows permits the transport of the failed bank material. Channel widening will continue until slope is reduced by channel lengthening. In the absence of channel widening, channel length will have to double for the design grade of 0.0007 to be achieved. (17 references)

48. M.D. Harvey, C.C. Watson 1988. "Channel Response to Grade-Control Structures on Muddy Creek, Mississippi." Regulated Rivers: Research and Management, Vol. 2. pp. 79-92.

Grade control structures are commonly employed to prevent bed degradation and concomitant bank instability of channelized reaches of rivers. A study of a 20-km reach of a coastal plains stream was conducted in 1985 to determine the effects of 12 rock-lined grade control structures that were installed between 1977 and 1983 prior to channel excavation. An allowable tractive stress method was used to determine the placement of grade control structures, and the gradients between them, for a trapezoidal-shaped channel designed to convey the 1-year recurrence interval peak flow. The design was successful in preventing bed degradation and bank erosion over the period of observation. However, unpredicted channel responses have occurred. Aggradation is apparent between control structures, and a two-stage compound channel has formed as a consequence of berm development, especially in the lower, older sub-reach. In the lower sub-reach, the vegetated berms have constricted the cross-sectional area at the design discharge (99 m^3/sec), and as a result, water-surface slope, shear stress, and unit stream power have increased. Bed material has become coarser and better sorted, which has increased shear intensity values. These unexpected changes are attributed to the lack of adequate consideration of the requirement for balance between sediment supply and transport in the allowable tractive stress method procedure. (24 references)

49. S. Hayashi, 1977. "On the Water Cushion between Sabo Dam and its Counter Dam." New SABO, No. 105, October, 1977 (in Japanese). pp. 1-10.

In case designing the counter-dam constructed to minimize the scouring of the downstream side of the main Sabo-dam, it is necessary to take into account the depth of scour by falling water. To calculate the depth of scour, the depth of the water cushion which varies with the influent hydraulic quantity between the main and counter-dam is needed. In this study, the characteristics of the jet flow of water fall, which is the cause of scour, and the depth of the water cushion between the main and counter-dam are examined. The flow in the water cushion is thought to be a two dimensional jet flow, however the hypothesis that the flow is one dimensional may be available for the actual analysis of the phenomena. The experiment performed to ascertain the characteristics of the flow in the scoured hole has shown good results and each depth of water cushion at their cross sections, calculated by applying the momentum theory and Albertson's two dimensional jet flow theory, has nearly agreed with the experimental results.

50. B.H. Heede, 1966 (reprinted 1970). "Design, Construction and Cost of Rock Check Dams." US Forest Service Research Paper RM-20, Rocky Mountain Forest and Range Experiment Station, Fort Collins, CO.

Loose-rock, wire-bound, single-fence, and double-fence dams and one type of headcut control were designed and installed in gullies on the White River National Forest. Motorized equipment proved to be suitable for installation of the structures. Gully control was least expensive with double-fence rock check dams. Higher double-fence structures are more economical on gully gradients steeper than 5 percent. The investigations indicated that rock check dams should maintain their place in modern gully control. (7 references)

51. J.L. Hicks, 1974. "Chicod Creek Watershed, Pitt and Beaufort Counties, North Carolina, Environmental Statement (Revised)" (see complete citation under US Department of Agriculture).

52. J.E. Hite, Jr., 1986. "Little Sioux Control Structure, Little Sioux River, Iowa; Hydraulic Model Investigation." Technical Report HL-86-5, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

The Little Sioux Project, located in Woodbury, Monona, and Harrison Counties, Iowa, consisted of remedial work on the channel of the Little Sioux River, three existing sills at the mouth of the river, and the construction of a channel control structure about 5.75 miles above the mouth. A model study of the original channel control structure was conducted to develop a satisfactory design for discharges up to 10,000 ft^3/s. Since the construction of the original control structure, the channel has degraded 11 ft, and flows exceeding 10,000 ft^3/s have occurred regularly. Flows exceeding the berm height scoured the side slopes causing the riprap to fail, and convergence of the concentrated flows from the right and left bank berm sections caused the development of a severe scour hole downstream of the stilling basin. High flows during the spring of 1983 caused the structure to fail so another model investigation was necessary to develop a design for the replacement structure and to determine methods to stabilize the area downstream of the structure and the channel side slopes.

Tests on a 1:25-scale hydraulic model of the replacement structure were conducted to develop the design. The model reproduced about 650 ft of topography upstream from the structure, the control structure, and 1,150 ft of topography downstream from the structure. Modifications to the original design were made to produce a structure that provided an acceptable headwater rating curve, and one with adequate energy dissipation in the stilling basin. A notched weir was developed that provided a desired range of headwater elevations for the expected discharges. The weir also produced velocities upstream and downstream from the low-flow notch for discharges less than 1,000 ft^3/s that were considered appropriate for upstream fish migration. Stable riprap designs were determined for the channel bottom downstream from the stilling basin and the channel side slopes.

53. J.E. Hite, Jr., G.A. Pickering, 1982. "South Fork Tillatoba Creek Drop Structures, Mississippi; Hydraulic Model Investigation." Technical Report HL-82-22, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

The South Fork of Tillatoba Creek, Mississippi, has experienced a relatively rapid stage of channel degradation accompanied by severe stream bank erosion. A two-stage, reinforced concrete grade control structure is proposed for construction by the US Department of Agriculture, Soil Conservation Service, on the South Fork of Tillatoba Creek. Tests were conducted on a 1:29-scale section model of the low-stage structure and a 1:25-scale model of the grade control structures to verify design criteria, evaluate the hydraulic performance of the structures, and make modifications to the design, where needed, to improve performance. The 1 :25-scale model reproduced both the high- and low-stage control structures, approximately a 500-ft length of the upstream approach, and a 1,100-ft length of the topography downstream from the structures. Flow conditions in the approach to the low-stage structure were improved with design modifications. Energy dissipation in the low- stage structure stilling basin was improved by the addition of a trajectory curve type of drop. Approach wing walls improved flow conditions in the high- and low-stage structure stilling basins. The widths of the exit channels for both the high- and low-stage structures were reduced from their original design. Improvements were made to flow conditions in the high-stage structure exit channel by eliminating the preformed scour hole, and stable riprap designs were determined.

54. F.M. Holly, T. Nakato, J.F. Kennedy, 1984. "Computer-Based Prediction of Alluvial River-Bed Changes." Proceedings of the Second Bridge Engineering Conference, Vol. 2, Transportation Research Board, National Research Council. Minneapolis, Minnesota. September 24-26, 1984. pp. 229-237.

Recent investigations and research programs at the Iowa Institute of Hydraulic Research have involved both the analysis and development of computer-based simulation techniques for alluvial riverbed evolution. The primary use of such techniques is in the prediction of riverbed aggradation and degradation caused by perturbations in the river's equilibrium geometry and sediment inflow supply over extended reaches. In this paper the mathematical basis of the problem is reviewed and several general numerical approaches and associated difficulties are described. Seven published programs are then described, and their performance when applied to three actual field situations is compared. The conclusions point out a critical dependence on field data and identify the need for further research in understanding physical mechanisms such as sediment sorting, armoring, scour, and deposition.

55. Z. Hou, R. Kahawita, 1988. "Nonlinear Hyperbolic System and Its Solutions for Aggraded Channels." Journal of Hydraulic Research. Vol. 26, No. 3. pp. 323-342.

A one dimensional nonlinear hyperbolic model that describes the unsteady sediment transport rate in an alluvial channel is presented. Solutions that are independent of other physical quantities involved have been obtained. This system, when used to evaluate the long time morphological changes due to constant upstream overloading, can be well approximated by one or two second order ordinary differential equations. An implicit numerical scheme has been developed for purposes of verification of the present nonlinear equation and its asymptotic solutions. In deriving the governing equation, a minimum of restrictive assumptions have been invoked. The results have been appraised by comparing them with the exact solutions to a linear model as well as with currently available experimental data.

56. I.W. Howe, 1955. "Aeration Demand of a Weir Calculated." Civil Engineering, Vol. 25, No. 5. American Society of Civil Engineers. pp. 59.

Aeration of a weir is necessary because of the entrainment of air beneath the nappe by the falling water. As the pressure is reduced, the discharge coefficient of the weir is significantly increased. Tests to determine the magnitude of the aeration demand were made using sharp-crested weirs varying in height from 2.0 to 3.5 ft, with heads up to 1.1 ft.

J

57. B.J. Jackson, 1974. "Stream Bed Stabilization in Enfield Creek, New York." New York Fish and Game Journal. Vol. 21, No. 1. pp. 32-46.

This study was undertaken to observe the performance of bed sills in stabilizing a high-gradient trout stream subjected to severe head cutting. Gabion structures were installed in Enfield Creek, New York, in 1967. A topographical survey of the streambed was made prior to installation and in 1968, 1970, 1971, and 1973. The sills were immediately effective in arresting head cutting and in accumulating gravel material, and they had a distinct stabilizing effect for almost 0.8 mile downstream. Gabion construction is discussed and a theory for partial failure of the two lower sills during the June 1972 flood is postulated.

58. W.F. Jaramillo, S.C. Jain, 1984. "Aggradation and Degradation of Alluvial-Channel Beds." Journal of the Hydraulics Division, ASCE. Vol. 110, No. 8, August, 1984. pp. 1072-1085.

A nonlinear parabolic model for nonequilibrium processes in alluvial rivers is presented. Analytical expressions for the characteristic parameters of relevant aggradation and degradation processes are derived. The validity and limitations of the model are assessed by comparing the analytical results with the available experimental data.

59. P.H. Jones, 1992. "Rock Riprap Grade Control Structure." ASAE paper 92-2563. 14 pp.

The Soil Conservation Service in Mississippi has gained experience in the design, construction, and performance of the Type 6 Grade Control Structure (GAS). This effort was based on model studies by the Waterways Experiment Station. The structure type results in an improved design and lower construction cost than the current TR-59 procedure. Recent riprap stability research in combination with a step backwater computer program, is incorporated in the design.

K

60. Kaiser Aluminum, 1980. Aluminum Structural Plate Drop Structures. 15 pp. 1980.

A commercial vender prepared handbook presenting design criteria for aluminum structural plate drop structures. The design criteria are based on specifications and design criteria developed by the USDA Soil Conservation Service.

61. G. Kalkanis, 1983. "Design and Analysis of Rock Chutes." Design Note No. 22 (see complete entry under US Department of Agriculture).

62. G. Kalkanis, 1985. "Hydraulics of Two-Stage Straight Drop Spillways" (see complete entry under US Department of Agriculture).

63. M.D. Kay, L. D. Medlin, 1986. "Gabion Chute Spillways for Grade Stabilization." ASAE paper 86-2132. 9 pp. (Mimeo handout)

Gabion chutes spillways offer a cost effective means of grade control or stabilization in numerous situations-particularly where unstable foundation conditions exist. Development, hydraulic design, and construction and installation procedures are discussed. A generalized design drawing and specification are presented.

64. A.M. Khashab, 1986. "Form Drag Resistance of Two-Dimensional Stepped Steep Open Channels." Canadian Journal of Civil Engineering. Vol. 13, No. 5, October, 1986. pp. 523-527.

Flow in rough steep open channels is mostly found in mountain streams and in flow overtopping protected weirs. In both cases, the energy of the flowing stream may be dissipated by artificial means so that the flowing water does not result in serious damage due to scour or erosion downstream of the main slope. The best way of achieving this purpose is to lead the flow over a series of steps.

65. F. Komamura, 1984. "Interrelationship Between Mountain-Stream Bed Slope and Variation of Bed Elevation above Sabo Dam." New SABO, Vol. 37, No. 2 (135), December, 1984 (in Japanese). pp. 6-13.

The stream-bed profile was developed by transfer of surface materials on the slopes. The process in developing the stream-slope was explained by a mathematical model which was formulated by the application of the theory of stochastic process of the transition probability of soil particles on the slopes. The solution of the model was examined to be applicable to represent the profiles of stream-slope and deposit-surface on the sabo dam which was formed by transfer of soil particles in the stream. It was described that the application of the theory of slope development was more appropriate than the application of Sternberg's law to explain the forming process of the longitudinal profile of a short stream on a mountain.

66. M.M. Kubo, 1983. "Placer Creek High-Velocity Channel and Debris Basin at Wallace, Idaho; Hydraulic Model Investigation." Technical Report 186-1, US Army Engineer Division, North Pacific Hydraulic Laboratory, Bonneville, OR.

A 1 :20-scale model of the Placer Creek high-velocity channel and debris basin was used to determine the adequacy of the proposed design. The model reproduced the entire 3,875-ft-long concrete-lined channel, the 420-ft-long debris basin, and approximately 620 ft of the South Fork Coeur d'Alene River at the exit of Placer Creek. Satisfactory flow conditions in the basin were achieved when the basin was deepened and a drop structure was added at the upstream end. The model verified that the basin design was effective in trapping debris. The original channel design proved to be satisfactory except in one area where two short reverse curves caused unacceptable waves in the channel. This condition was remedied by realigning the channel using a straight-line transition. Movable-bed studies showed that high Placer Creek discharges would develop a large scour hole in the South Fork Coeur d'Alene River at the exit of the high-velocity channel. Although not tested in the model, a grouted riprap section was included in the prototype to minimize the scour potential.

L

67. E.M. Laursen, 1984. "Assessing the Vulnerability of Bridges to Floods." Proceedings of the Second Bridge Engineering Conference. Minneapolis, MN. Transportation Research Record 950. National Research Council. Washington, DC. Vol. 2. pp. 222-229.

The capacity of both new and old bridges to withstand scour at their foundations and any other flow phenomena that could lead to failure needs to be examined. The problem is somewhat different in the two cases because a new bridge should be designed for the maximum flood to be expected, and there is ample opportunity in the design process to suggest foundations--or even bridge configurations--that may lead to safer, less costly bridges. Making existing bridges less vulnerable is likely to be difficult, awkward, and costly. However, even old bridges can be valuable--as can be discovered after they are lost--but the cost of remedial measures for the maximum expected flood may be more than can be justified. The prediction of scour at bridge foundations is a three-step procedure: (a) the establishment of the flood magnitude-frequency relationship, (b) the conceptualization and analysis of the flow characteristics of floods that might occur during the life of the bridge, and (c) the prediction of scour. The first step needs evidence of the maximum flood that should be expected; the second step is the most difficult as a general rule; the third step is likely to raise questions about scour that have not yet been answered adequately. As a result of the Silver Bridge failure, visual examination of bridges for structural integrity has become routine. Despite occasional spectacular failures like the Interstate bridge in Connecticut, there are probably more bridges lost in floods than from structural inadequacy. The assessment of the vulnerability of existing bridges to floods is also needed and would pay dividends. Recent research, sponsored by the Arizona Department of Transportation and the FHWA, has resulted in relationships for prediction the scour at the toe of a vertical wall and at the toe of a sloping sill. On the basis of the depth of scour, the structural form, and the ease of adding to the sill structure if need be in the future, the sloping sill is the preferred solution. Recent unsponsored student research indicates that the previous solution for sizing riprap was too conservative. Both of these studies are aids to the engineer seeking ways to make existing bridges less vulnerable. (29 references)

68. E.M. Laursen, M.W. Flick, 1983. "Predicting Scour at Bridges: Questions Not Fully Answered--Scour at Sill Structures." Report No. FHWA/AZ-83/184-3, Arizona Department of Transportation, Phoenix, AZ.

Degradation of the streambed is likely to be the reason for constructing sill structures. A discussion of the degradation phenomenon is included to serve as a guide to evaluate to what extent degradation might be a threat to a bridge, culvert, or highway. (23 references)

69. E.M. Laursen, M.W. Flick, 1983. "Scour at Sill Structures." Final Report to Arizona Department of Transportation, AZ HRP-1-19 (184), Arizona Transportation and Traffic Institute, The University of Arizona, Tucson, Arizona, 1983.

The scour at the toe of a vertical wall and at the toe of a sloping sill were investigated experimentally and analytically. Approximate relations for predicting the ratio of the scour depth to the critical depth were obtained for the two geometries. For the vertical wall, the sediment scoured our left in suspension, and the parameters needed to describe the scour phenomenon were the ratio of the critical velocity to the fall velocity and the drop in water surface in ratio to the critical depth. For the sloping sill, which is the recommended geometry, the sediment scoured our left as bed load, and the parameters needed to describe the scour phenomenon were the critical depth/sediment size ratio and the ratio of the size of the riprap protecting the sill slope to the critical depth. Degradation of the stream bed is likely to be the reason for constructing sill structures. A discussion of the degradation phenomena is included to serve as a guide to evaluate to what extent degradation might be a threat to a bridge, culvert or highway.

70. E.M. Laursen, M.W. Flick, B.E. Ehlers, 1986. "Local Scour at Drop Structures." Proceedings of the Fourth Federal Interagency Sedimentation Conference. Las Vegas, Nevada. March 24-27, 1986, Vol. I. pp. 4.60-4.69.

The scour at the toe of a vertical wall and at the toe of a sloping sill were investigated experimentally and analytically. Approximate relations for predicting the ratio of the scour depth to the (energy) critical depth were obtained for the two geometries. For the vertical wall, the sediment scoured out left in suspension, and the parameters needed to describe the scour phenomenon were the ratio of the (energy) critical velocity to the fall velocity and the drop in water surface in ratio to the critical depth. For the sloping sill, which is the recommended geometry, the sediment scoured out left as bed load, and the parameters needed to describe the scour phenomenon were the critical depth/sediment size ratio and the ratio of the size of the riprap protecting the sill slope to the critical depth. A follow-up study of flow and scour characteristics for a vertical drop followed by an apron resulted in rules for determining the length of apron required and the range of tailwater for proper flow. (7 references)

71. W.M. Linder, 1963. "Stabilization of Streambeds with Sheet Piling and Rock Sills." Proceedings of the Federal Interagency Sedimentation Conference. Subcommittee on Sedimentation, ICWR. Jackson, MS. Miscellaneous Publication No. 970, Paper No. 55, US Department of Agriculture, Agricultural Research Service. pp. 470-484.

This paper summarizes the conditions that led to the development of the channel stabilization structure for the Floyd River flood control project in Sioux City, IA, and describes the procedures followed in two model studies conducted at the University of California at Berkeley and the University of Iowa, Iowa City.

72. W.M. Linder, 1976. "Designing for Sediment Transport." Water Spectrum Vol. 8, No. 1. pp. 36-43.

Stream channelization is frequently employed without consideration of sediment transport characteristics of the stream, or of the extent to which the stream's natural balance will be disturbed. Erosion and sedimentation damage are the critical adverse effects. Techniques should be employed which consider sediment transport characteristics and stream equilibrium. In this way, the reactions of a stream to man-made changes can be minimized. Some of the techniques that should be considered are (1) composite channel designs, (2) extensive use of levees, and (3) grade control structures. The ultimate cost resulting from adverse effects of traditional channel modification is greater than the cost of a design that recognizes the influence of sediment transport.

73. W.C. Little, 1983. "Stilling Basin Design for Low Drop Structures." ASAE Paper No. 83-2115. 10 pp. (Mimeo handout)

Rectangular shaped stilling basins for low drop grade control structures were investigated. Design criteria for the length and width of basin are given to supplement other previously established design criteria.

74. W.C. Little, R.C. Daniel, 1982. "Design and Construction of Low Drop Structures." Proceedings of the ASCE Conference Applying Research to Hydraulic Practice. Jackson, MS. August 17-20, 1982. Peter E. Smith, ed.. pp. 21-31.

Hydraulic design criteria for low drop channel grade control structures are reviewed. Guidelines and experiences in structural design, layout, and construction are given. The hydraulic performance of a field structure is discussed. These low drop structures overcome the problems associated with low drop structures at high discharges where undulating waves are normally generated. A rock-lined stilling basin with either a baffle pier or plate is provided to dissipate the energy through the drop. (6 references)

75. W.C. Little, J.B. Murphy, 198la. "Stream Channel Stability; Comprehensive Report: Project Objectives 1 thru 5; Appendix A: Evaluation of Streambank Erosion Control Demonstration Projects in the Bluff Line Streams of Northwest Mississippi." prepared for US Army Corps of Engineers, Vicksburg District, Vicksburg, MS, under Section 32 Program, Work Unit 7, USDA-ARS National Sedimentation Laboratory, Oxford, MS.

This report presents an evaluation of some of the most commonly used types of channel bed and bank revetments used on bluff line streams of the Yazoo Basin. Specific revetment techniques used on Goodwin Creek, Johnson Creek, Hotopha Creek, Tillatoba Creek, and Perry Creek watersheds were used to illustrate successes and failures. A complete section is devoted to each of these watersheds and includes the watershed description, geology, soils, and evaluation of performance of selected revetment techniques. Special attention is given in this report to a newly developed, low-cost grade control structure. During this project, 12 of these grade control structures were built and their performance was evaluated for this report. Generalized engineering design criteria for these low drop grade control structures are given in Appendix B of the general report. (55 references)

76. W.C. Little, J.B. Murphy, 1981b. "Stream Channel Stability; Comprehensive Report: Project Objectives 1 thru 5; Appendix B: Model Study of the Low Drop Grade Control Structures." prepared for US Army Corps of Engineers, Vicksburg District, Vicksburg, MS, under Section 32 Program, Work Unit 7, USDA-ARS National Sedimentation Laboratory, Oxford, MS.

This report presents the results of hydraulic model tests for low drop grade control structures. The results are presented in dimensionless relationships. Tentative design criteria are formulated for the design of low drop grade control structures with baffle energy dissipation devices. A method is given to determine the size of the stilling basin and the size and placement of a baffle pier or baffle plate in the basin to achieve optimum flow conditions in the downstream channel. (8 references)

77. W.C. Little, J.B. Murphy, 1982. "Model Study of Low Drop Grade Control Structures." Journal of the Hydraulic Division. ASCE. Vol. 108, No. HY10. pp. 1132-1146.

Low drop grade control structures were subjected to model studies for the purpose of designing structures for installation in creeks in Mississippi and Arkansas. These structures control the severe channel degradation seen in alluvial valley streams in which head cuts or over falls of 0.3-1.5 m in height progress upstream, causing bed scour, bank slumps and slides, and channel widening. Low drop is defined as a hydraulic drop with a relative drop height equal to or less than 1; a high drop has a value greater than l. The relative drop height is defined as the difference in elevation between upstream and downstream channel beds divided by the critical depth. Using a model basin, tentative design criteria were developed for stilling basins for low drops with energy dissipation devices. A method is also given to determine the size and placement of a baffle pier or plate to obtain optimum flow conditions downstream. (7 references)

78. J-Y Lu, H.W. Shen, 1986. "Analysis and Comparisons of Degradation Models." Journal of the Hydraulics Division, ASCE, Vol. 112, No. 4, April, 1986. pp. 281-299.

Construction of a dam is one of the most common causes of channel degradation. The major differences among several well-known mathematical degradation models are classified. The analytical degradation model is based on the solution of a diffusion equation, which is converted from the governing equations using kinematic wave assumption. Most of the explicit models are similar, but differ in terms of the calculation of longitudinal sediment distribution to update the channel bed profiles at each time step. The explicit model proposed by Gessler is simpler than other explicit models. Laboratory data previously collected at Colorado State University for degradation study in a prismatic channel with nearly uniform sediment sizes are used to compare the results predicted by different explicit and implicit models. Gessler's model produces rather accurate results for flow conditions with high Froude number (but still subcritical flow) in a prismatic channel. Results from the central difference scheme as used in HEC-6 for the longitudinal sediment distribution appears to agree well with the collected flume data. The difference between the actual sediment load and the sediment transport capacity within the "distance of concentration recovery" has to be considered for modeling the degradation process. Comments are made about the applicability and limitations of different models. The variation of roughness during degradation is also analyzed.

M

79. J.J.R. Matos-Silva, 1986. "Kinematic-Wave Analysis of River Responses to Imposed Disequilibria." Ph.D. Thesis. The University of Iowa. December, 1986.

The objective of the research reported herein was development of simple, one-dimensional, closed-form, analytical models, based on a kinematic-wave framework (Lighthill and Whitham 1955) and on validated mathematical expressions of the constituent phenomena, for analysis of river-bed responses to imposed disequilibria, such as variations in the upstream sediment input. The potential and limitations of kinematic-wave theory applied to degradation and aggradation processes were analyzed. The kinematic-wave models include coarsening and armoring of degrading stream beds composed of nonuniform sediment.

80. R.W. McCarley, J.J. Ingram, B.J. Brown, A.J. Reese, 1990. "Flood-Control Channel National Inventory." Miscellaneous Paper HL-90-10, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

A US Army Corps of Engineers-wide survey of flood-control project design procedures and related experiences was conducted in 1985 to record the following: (1) specific information about various streams and promising improvement techniques, design methods used in the past, centers of experience for certain type projects, points of contact by name, and stream types existing in each Corps Division; (2) problems and noteworthy experiences pertaining to project design, environmental issues, local cooperation, Corps District operation and maintenance activities, and project review; and (3) insight into future research and guidance needs for bank protection (particularly riprap), grade control structures, and stable channel design in general. (41 references)

81. McLaughlin Water Engineers, Ltd. 1986. "Evaluation of and Design Recommendations for Drop Structures in the Denver Metropolitan Area." Report prepared for Urban Drainage and Flood Control District, Denver, CO.

This document presents an evaluation of and design guidance for drop structures in the Denver metropolitan area for the Urban Drainage and Flood Control District. The study scope focused on analyzing drops having design flows up to 15,000 ft^3/s, with primary emphasis on grass-lined channels having design flows up to 7,500 ft^3/s. Flows less than 500 ft^3/s were addressed for small drops, trickle channels, and local drainage "rundowns" for conveying minor tributary flows into major drainage ways. In the discussion, pertinent literature reviewed during the project is presented. References applicable to various topics are denoted. Design guidance for the following basic categories is presented: (1) VRR: Vertical Riprap Drop, (2) SLR: Sloping Large Riprap Drop, (3) GSB: Grouting Sloping Boulder Drop, (4) BC: Baffle Apron (Chute) Drop, and (5) VHB: Vertical Hard Basin Drop. This report presents economic evaluations for these five drop categories, as well as the District's present sloping riprap drop design. These evaluations include both capital costs and maintenance costs. Included is an economic efficiency relationship which should be useful to

designers. The efficiency relationships reflect the economy of scale and economic considerations for various drop heights and design flow rates. (67 references)

82. McLaughlin Water Engineers, Ltd. 1989. "Evaluation of and Design Recommendations for Drop Structures in the Denver Metropolitan Area." Addendum and Errata, report prepared for Urban Drainage and Flood Control District, Denver, CO.

This document presents an evaluation of and design guidance for drop structures in the Denver Metropolitan area for the Urban Drainage and Flood Control District. This study will likely prove applicable to similar structures in other regions.

83. E.S. Melsheimer, J.L. Grace, Jr., 1966. "Fremont Drop Structure and Friction Channel, Sandusky River, Ohio; Hydraulic Model Investigation." Technical Report 2-752, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Investigations were conducted to determine the adequacy of a drop structure or a friction channel to provide flood control and fish passage facilities in a reach of the Sandusky River near Fremont, OH. The drop structure investigation was concerned with its discharge characteristics, performance of the stilling basin, the openings required for fish passage facilities, and various plans of protective stone. Modifications of the original design structure improved hydraulic performance and reduced construction costs. These modifications included lengthening the stilling basin and depressing the apron, replacing the concrete pavement downstream of the end sill with stone, providing satisfactory fish passage facilities by means of an alternate weir arrangement, reducing the width of the channel below the structure, and using a trapezoidal rather than a rectangular structure. Four types of friction channels were investigated, one of which provided the desired stages, velocities, and conditions required for fish passage.

84. E.S. Melsheimer, T.E. Murphy, 1965. "Drop Structure, Cayuga Inlet, Cayuga Lake, New York; Hydraulic Model Investigation." Technical Report 2-709, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

The drop structure proposed for Cayuga Inlet was studied on a 1:20-scale model to determine the adequacy of the structure. Particular emphasis was placed on selection of a basin which would operate satisfactorily with tailwater depths greater than that required for a satisfactory hydraulic jump. The original design structure proved unsatisfactory, as it did not provide adequate water levels upstream from the structure; addition of a 3-ft-high crest sill was required. Use of a 12-ft-radius abutment wall instead of the wing walls of the original design reduced draw down at the abutments, improved basin action, and reduced construction costs. Baffle piers were not required in the stilling basin, and it was found that the length of the apron could be reduced from 60 to 50 ft without a noticeable effect on energy dissipation.

85. K. Mizuyama, T. Bando, 1985. "Arrangement of Sabo Structures Using Hydraulic Model Tests and Analytical Methods." New SABO, Vol. 38, No. 2 (139), July, 1985 (in Japanese). pp. 6-11

We have not yet had a rational method of determining the height and site of sabo-dams and alignment of channels. when the condition is complex, hydraulic model experiments have been executed to find the optimal sabo structure arrangement. Computer models may be required first if the condition is very wide or the dimension of the structures changeable. This paper includes the method to obtain the final sabo structure arrangement plan with hydraulic model tests and river bed change calculations.

86. W.L. Moore, 1943. "Energy Loss at the Base of a Free Overfall." Transactions, American Society of Civil Engineers, Paper No. 2204, with discussion by Messrs. Merit P. White, Boris A. Bakhmeteff and N. V. Feodoroff, Carl E. Kindsvater, J. E. Christiansen, L. Standish Hall, Hunter Rouse, and Walter L. Moore, Vol. 108. pp. 1343-1392.

Experimental studies were made of a free over fall with a view to obtaining information that would be of value to designers of hydraulic structures. Detailed laboratory measurements showed that the energy losses at the base of a fall were of appreciable magnitude and hence must be considered in hydraulic design. These measured energy losses were applied in the development of a rational formula for calculating the height of the jump below a fall. Limited information was also obtained on the length characteristics of the jump and on the effect of submergence of the jump on energy dissipation. The presence of standing water behind the fall is explained, and its height is calculated by application of the momentum equation. (14 references)

87. B.T. Morris, D.C. Johnson, 1943. "Hydraulic Design of Drop Structures for Gully Control." Transactions. American Society of Civil Engineers. Paper No. 2198, with discussion by John Hedberg, L. Standish Hall, J. E. Christiansen, Walter T. Wilson, N. A. Christensen and Dwight Gunder, Boris A. Bakhmeteff and Nicholas V. Feodoroff, G. H. Hickox, and B. T. Morris and D C. Johnson, Vol. 108. pp. 887-940.

In the stabilization of gullies, small overflow dams are used to retain silt and to control the stream grade. These dams are simple drop structures similar to those used in irrigation canals. In this paper the development of rules for the proportioning of such dams is described in terms of the hydraulic requirements for structure performance. The formulas included in the design rules are presented graphically for convenience in application. These rules are based on the accumulated experience of engineers in irrigation and soil conservation work and on the results of a series of laboratory test programs. (15 references)

88. T.E. Murphy, 1971. "Control of Scour at Hydraulic Structures." Miscellaneous Paper H-71-5, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

A case is made for providing for or pre-forming a scour hole in which flow from a hydraulic structure can expand and dissipate its excess energy in turbulence rather than in a direct attack on the channel boundaries. Examples are given which demonstrate that riprap schemes providing for flow expansions make it feasible to stabilize the channels with rock of an economical size and provide factors of safety against riprap failure and costly maintenance.

89. T.E. Murphy, 1967b. "Control Structure, Little Sioux River, Iowa: Hydraulic Model Investigation." Technical Report 2-762, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

The control structure in the Little Sioux River in Monona County, Iowa, was designed to provide non-scouring velocities upstream in the channel and on the channel berms between the levees. The structure consists of a rectangular drop in the central channel flanked by rock sills on the berms. Tests on a 1:30-scale hydraulic model were concerned with capacity of the structure, effectiveness of the concrete drop, and adequacy of the rock protection for the sills, channel, and berms.

90. T.E. Murphy, 1967a. "Drop Structures for Gering Valley Project, Scottsbluff County, Nebraska, Hydraulic Model Investigation." Technical Report 2-760, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Tests were conducted on a 1:12-scale model of a rectangular drop structure designed to stabilize channel beds and minimize bank erosion in the Gering Valley drainage system. The majority of the tests were conducted on a 33-ft-wide structure with a drop height of 5 ft. Discharges up to a maximum of 6,000 ft^3/s were observed. Verification of generalized data was accomplished by tests on a structure with a 10-ft drop height. Of primary concern were development of the optimum dimensions for the various elements of the structure and determination of riprap requirements in the vicinity of the structure.

91. C.T. Myers, Jr., 1982. "Rock Riprap Gradient Control Structures." Proceedings of the ASCE Conference Applying Research to Hydraulic Practice. Jackson, MS. August 17-20, 1982, Peter E. Smith, ed.. pp. 10-20.

The structure consists of a riprap prismatic channel with a riprap transition at each end designed to flow within the subcritical range. One design feature is that the specific energy of the flow at design discharge is constant throughout the structure and is equal to the specific energy of the flow in the channel immediately upstream and downstream from the structure. Thus, for the design discharge the dissipation of hydraulic energy in the structure is at the same rate as the energy gain due to the gradient. The Soil Conservation Service in Mississippi has constructed 14 rock riprap gradient control structures. in Tippah County, 12 are located on Muddy Creek and l on Tippah River. One is located on Running Slough Ditch in Panola County. The design capacity of the structures ranges from 622 ft^3/s (17.4 m^3/sec) to 20,600 ft^3/s (576.8 m^3/sec). These structures have not been model tested, but they have been inspected in the field periodically. All the structures have performed as designed with regard to establishing a stable gradient in the channel in which they are constructed. In May 1980, field surveys revealed a deep scour hole forming at the downstream end of the exit transition on the three structures farthest downstream on Muddy Creek. However, no damages have occurred to the structures themselves. Measurements of point velocity and depth at various cross sections within the prototype structure to determine velocity distribution and roughness were performed. (2 references)

N

92. T. Nakato, 1989. "A Review of International Literature of Design Practice and Experience with Low-Head Alluvial-Channel Grade-Control Structures." Prepared by the Iowa Institute of Hydraulic Research for the US Army Engineer Waterways Experiment Station, Vicksburg, MS.

The principal objective of this investigation was to survey the international literature and to canvass the principal hydraulics design and testing organizations around the world for current information on the different types of low-head drop structures which have been developed and installed, and to review the experience with these structures. The supplemental objective as to review the applicability of existing analytical models and so-called "kinematic analysis" to small streams, which are grade-controlled with single or cascaded low-head drop structures to obtain rapid yet reasonably accurate predictions of the equilibrium bed profiles of the steams. (73 references)

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93. N.R. Oswalt, 1978. "Model Studies of the Portugues and Bucana Rivers Channelization, Puerto Rico; Hydraulic Model Investigation." Technical Report H-78-3, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Two 1:30-scale physical hydraulic models of the Portugues and Bucana Rivers were used to determine the adequacy of the original designs for the flood control channelization project through the city of Ponce, Puerto Rico. The proposed channelization included trapezoidal to rectangular channel transitions, stilling basins, and drop structures in the high-velocity channels. A 3,300-ft (prototype) channel length was used to study the transitions and stilling basins including riprap stability downstream from the stilling basins. A 1,200-ft (prototype) channel length was used to study the drop structures and the adjacent riprap protection requirements. Test results indicated that the original design with certain modifications would effectively transmit all expected flood releases from the proposed Portugues and Cerrillos Dams. Modifications to transitions at entrances to the high-velocity channel reaches were streamlined within the original right-of-way to reduce surface turbulence and standing waves. Geometry of the original stilling basins was altered to prevent the oblique hydraulic jumps, and the end sill heights were lowered to reduce the water-surface draw down, surface roller waves, and high bottom velocities downstream of each basin.

94. N.R. Oswalt, J.F. George, G.A. Pickering, 1975. "Fourmile Run Local Flood-Control Project, Alexandria and Arlington County, Virginia." Technical Report H-75-19, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

The Fourmile Run flood-control project will provide protection for the city of Alexandria and Arlington County, VA, from flooding of the Fourmile Run channel. The proposed plan for containing flood flows consists of eliminating existing constrictions in the channel; excavating and widening the channel; providing adequate bank slope protection; and constructing energy dissipaters, grade control structures, and flow dividers. The investigation was conducted with a 1:30-scale model that reproduced approximately 5,700 ft of the Fourmile Run channel and 400 ft of the Long Branch tributary. The existing bridges and vertical walls, as well as the proposed improved channel with bank slope protection and hydraulic structures, were reproduced in the model. Tests encompassed flow conditions at the bridges and at the confluence of Long Branch, water-surface elevations, riprap and gabion stability, and performance of the hydraulic structures.

P

95. I. Park, S.C. Jain, 1986. "River-Bed Profiles with Imposed Sediment Load." Journal of the Hydraulics Division, ASCE, Vol. 112, No. 4, April, 1986. pp. 267-280.

The rate and extent of bed aggradation resulting from sediment overloading are determined by means of computer-based numerical experiments. A nondimensional momentum equation, along with continuity equations for water and sediment of unsteady flow for a wide, prismatic channel, are solved numerically using recently developed relations for sediment discharge and friction factor. Numerical experiments are executed for a range of values of nondimensional input data. Principal dependent variables, such as changes in bed levels from equilibrium levels and the length of aggradation, determined form the computer experiments are correlated by multiple linear regression analysis to the independent nondimensional variables. Numerical results show that the sediment diffusion coefficient is a function of the rate of sediment overloading. The agreement between the numerical results and the available experimental data is satisfactory.

96. B.C. Phillips, A.J. Sutherland, 1986. "Diffusion Models Applied to Channel Degradation." Journal of Hydraulic Research. Vol. 25, No. 3, 1986. pp. 179-191.

The application of the diffusion model to bed degradation due to the sudden cessation of sediment supply to an alluvial system has been investigated and limitations of the model noted. Experiments indicate that local maximum scour depths do not occur at the upstream end of an erodible reach. This was accounted for by modifying the diffusion model to include a mobile upstream boundary scheme. The modified model was calibrated using bed load sediment transport data and diffusion coefficient values of best fit obtained. Significant differences between calibrated and predicted diffusion coefficient values were obtained. Measured and predicted bed profiles were also compared and conclusions about the applicability of the diffusion model to bed degradation drawn.

97. D.M. Patrick, L.M. Smith, C.B. Whitten, 1982. "Methods for Studying Accelerated Fluvial Gravel-Bed Rivers: Fluvial Processes. Engineering and Management, John Wiley & Sons, New York. pp. 783-815.

To develop and manage a river basin and fluvial system effectively, it is necessary to identify and minimize the adverse effects of existing structures and activities and to predict and take into account the potential adverse effects of existing structures and activities and to predict and take into account the potential adverse effects of proposed schemes. This requires data on the factors that control the mechanics of the fluvial system. These data should cover the basin characteristics, relations between geomorphology and river mechanics, erodibility of the drainage basin, and temporal effects. In this paper some of the techniques and methods for studying accelerated fluvial change are described. Particular emphasis is placed on the importance on basin characteristics and on geomorphological processes to project studies. Three case studies are used as examples of the applications of the techniques and methods described. (29 references)

98. G.A. Pickering, 1966. "Drop Structures for Walnut Creek Project, Walnut Creek, California; Hydraulic Model Investigation." Technical Report 2-730, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Walnut Creek project will provide for enlargement and rectification of the existing channels of Walnut, Lower San Ramon, and Las Trampas Creeks. Three grade control structures and one energy dissipating structure will be used to reduce velocities and dissipate excessive energy from flood flows. Model investigations were conducted on 1:20-scale models of drop structures 2 and 3. Tests were concerned with determining the optimum size and configuration of the stilling basins for these structures, the stability of the riprap downstream from the stilling basins, and the adequacy of the inlet transition upstream from drop structure 2.

99. G.A. Pickering, 1983. "Big Creek Flood-Control Project, Cleveland, Ohio; Hydraulic Model Investigation." Technical Report HL-83-7, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Tests were conducted on a 1 :40-scale model of Big Creek to investigate the hydraulic performance of a proposed floodway channel and a section of modified channel. The model reproduced the entire proposed floodway channel, a portion of the existing Big Creek main channel, and a section of modified channel downstream from the existing channel. The original design gabion drop structures used in the floodway to reduce the grade along the channel caused flow to concentrate near the middle of the channel, resulting in high velocities. Revisions to these drop structures resulted in good flow conditions throughout the floodway channel. A concrete transition between the downstream end of a three-barrel conduit and the modified channel caused flow to concentrate in the channel, which resulted in movement of the riprap immediately downstream from the transition and along the left bank. The concrete transition was removed, and the area was shaped with riprap. This resulted in satisfactory flow conditions in the modified channel.

100. Pima County Department of Transportation and Flood Control District, 1985. "Soil Cement Applications and Use in Pima County for Flood Control Projects." Tucson, AZ.

The primary use of soil cement in Pima County has been to reduce the erosion of unstable natural channel banks. Soil cement design specifications, engineering analysis, construction techniques, and current research are explained along with examples of the performance of soil cement bank stabilization during the October 1983 flood. while soil cement has a long record of satisfactory service as a paving material for highways, streets, and airports, it has also been successfully used in energy and water resource projects. Applications include slope protection, seepage control, and foundation stabilization. The listed advantages of soil cement include low cost, ease of construction, and the convenient utilization of local or in-place soil, thus making soil cement applications economical, practical, and environmentally attractive. Within Pima County, soil cement has been used for channel bank stabilization, grade control structures, channel bed protection, detention basins, landfill protection, bridge abutment protection, and as base material for highways. To date, soil cement has proven to be the only effective method of bank stabilization on the major river system in Pima County.

R

101. W. Rand, 1955. "Flow Geometry at Straight Drop Spillways." Proceedings. American Society of Civil Engineers, Paper No. 791, Vol. 81. pp. 1-13.

The flow pattern at a straight drop spillway can be described by a number of characteristic length terms: the drop length, that is, the distance from the vertical drop wall to the toe of the non-submerged nappe; the depth of flow at the toe of the nappe; the length of the hydraulic jump if it begins at the toe of the nappe; the depth of flow downstream from this jump; and the depth of the under-nappe pool between the drop wall and the nappe. All these values are represented in this paper as functions of the discharge and of the height of the drop. The results are given by a collective plot of dimensionless terms. Two geometrical properties of the flow pattern are established, consisting of practically constant relationships between some of the terms. The determination of flow geometry is important for the design of straight drop stilling basins. (5 references)

102. W. Rand, 1970. "Sill-Controlled Flow Transitions and Extent of Erosion." Journal of the Hydraulics Division. ASCE. Vol. 96, No. HY4. pp. 927-939.

Sill-controlled flow transitions in open channels, dependent on the geometry of a rigid (fixed-bed) boundary, have been described earlier. This investigation deals with the same flow transitions where the channel downstream of the sill is erodible. A similarity concept for erosion, to be confirmed by experimental evidence, will be established as an extension to the similarity criteria valid for a rigid boundary. As a result, prediction of the extent of erosion is expected to become possible for a wide variety of sill-controlled flow transitions, including the natural hydraulic jump, and flow transitions present in the hydraulic jump stilling basins. (6 references)

103. T.J. Rhone, 1971. "Studies to Determine the Feasibility of a Baffled Apron Drop as a Spillway Energy Dissipater, Conconully Dam Spillway, Okanogan Project, Washington." Report REC-ERC-71-29, Bureau of Reclamation, Denver, CO.

The existing spillway structure at Conconully Dam, Washington, was determined to be structurally unsafe and incapable of discharging the design flood. Installation of a conventional hydraulic jump stilling basin or flip bucket to handle the design flood was impractical because of poor foundation conditions. Preliminary investigations showed how, if the allowable unit discharge of a baffled apron drop could be increased from 60 $ft^3/s/ft$ to roughly 80 $ft^3/s/ft$, such a structure could be built on sound rock. hydraulic model studies were performed to confirm a design for a battled apron drop basin on a unit discharge of 77.7 $ft^3/s/ft$. The tests indicated that the higher capacity structure was an effective and safe energy dissipater, and could handle unit discharges up to twice the design discharge. The effect of baffle pier location on the reservoir elevation for maximum discharge was determined. An optimum configuration for the channel bed downstream of the concrete apron was developed to prevent erosion of the apron. (1 reference)

104. J.S. Ribberink, J.T.M. Van der Sande, 1985. "Aggradation in Rivers due to Overloading - Analytical Approach." Journal of Hydraulic Research, Vol. 23, No. 3, 1985. pp. 273-283.

The problem of aggradation in a river due to overloading is tackled with a mathematical model consisting of a set of one-dimensional (in space) basic equation in which the water motion is assumed to be quasi-steady and the sediment transport is determined by local conditions Analytical solutions are presented of a linearized simple wave model, parabolic model and the more general hyperbolic model. For large disturbances in the sediment transport an adapted solution of the hyperbolic model is obtained. Numerical computations with the complete set of basic equations learn that this solution yields better results for large and small disturbances. A laboratory experimental verification of this adapted solution shows a satisfactory agreement between the measured and calculated aggradation.

105. C.E. Rice, K.C. Kadavy, 1987. "Energy Dissipation for a SAF Stilling Basin." Applications of Engineering in Agriculture. 3(1):52-56.

Results of a physical model study to determine the design for a pre-shaped, riprap energy dissipation pool at the exit of a SAF stilling basin located on an irrigation canal in southwestern Colorado are presented. Fine, non-cohesive bed material was used to establish the form of the scour patterns used to develop the geometry of the pre-shaped energy dissipation pool.

106. C.E. Rice, K.C. Kadavy, 1989. "Scour Protection at the Straight Drop Spillway." Proceedings of the ASCE National Conference on Hydraulic Engineering. New Orleans, LA. August 14-18, 1989. Michael A. Ports, ed.. pp. 7-12.

Tests were conducted to determine the scour and the effect of tailwater on scour downstream of the straight drop spillway stilling basin. Preliminary results are presented on performance with three riprap sizes. Spillway performance with and without wing-walls is discussed. (2 references)

107. C.E. Rice, K.C. Kadavy, 1991. "Riprap Design Downstream of Straight Drop Spillways." Transactions of the American Society of Agricultural Engineers. 34(4):1715-1725.

Tests made to determine the scour downstream of the straight drop spillway stilling basin and the size and placement of riprap to ensure stability of the structure are described in the paper The problem and need for making the tests are discussed. Results are presented on the performance of the spillway with square and rounded abutments at the entrance to the stilling basin and with basins shorter than those recommended by the original design criteria. Criteria are presented to select the size and placement of riprap downstream of straight drop spillway stilling basins to ensure the integrity of the structures.

108. C.E. Rice, K.C. Kadavy, 1992. "Riprap Design Upstream of Straight Drop Spillways." Transactions of the American Society of Agricultural Engineers. 35(1): 113-119.

Tests were made to determine the size and placement criteria for riprap upstream of straight drop spillways, with rounded and square entrance abutments. Riprap elevations used were 0.0(D_{50}), 1.0(D_{50}), and 2.0(D_{50}) below the elevation of the spillway weir crest. At 1.0(D_{50}) and 2.0(D_{50}) below the weir crest elevation, substantial scour occurred only in the vicinity of abutments. At the 0.0(D_{50}) crest elevation, substantial scour occurred both in the center area upstream of the weir crest and in the vicinity of the abutments. Size of riprap required to prevent scour decreased as the tailwater surface increased above the elevation where it affected the upstream water surface elevation. Test results can be used to design size and placement of riprap that will ensure spillway integrity.

109. C.E. Rice, K.C. Kadavy, 1992. "Riprap Design for SAF Stilling Basins." Transactions of the American Society of Agricultural Engineers. 35(6):1817-1825.

Physical model studies were conducted to determine criteria for the size and placement of riprap downstream of SAF stilling basins to ensure basin integrity. Relationships are presented to determine the depth, width, and length for preformed scour holes as a function of the Froude number and riprap size. Results show that the riprap size required for stability increases exponentially with Froude number. Smaller riprap is required for stability if the riprap is recessed below the elevation of the basin floor. For level riprap placement, larger riprap is required for stability when the riprap is placed at the end sill elevation compared to placement at the basin floor elevation.

110. D. Richard, R.K. Williams, R. Swanson, 1987. "Low Head Dam Safety Studied with Hydraulic Models." Proceedings of the ASCE National Conference on Hydraulic Engineering. Williamsburg, VA. August 1-7, 1987. Robert M. Ragan, ed.. pp. 528-533.

Use of physical models to study the hydraulic jump or submerged roller action created as water overflows low head dams provides an opportunity to develop design criteria for various structure configurations for small sums of money. Low head dams, although very attractive to visitors, can be treacherous due to the unseen, violent and submerged hydraulic undercurrent. Several models of the proposed low head dam for Riverside Park in Grand Forks, North Dakota were constructed and tested in the hydraulics laboratory at North Dakota State University. A cascade structure, although the most expensive, was recommended due to safety considerations. Video tapes of the model studies further allowed public education and understanding of the potential danger in the various design alternatives.

111. K.M. Robinson, 1989. "Overfall Stress and Pressure Distributions." Proceedings of the ASCE National Conference on Hydraulic Engineering. New Orleans, LA. August 14-18, 1989. Michael A. Ports, ed.. pp. 943-948.

The magnitude and distribution of hydraulic shear stress and pressure on the boundary of a straight drop over fall are presented for multiple over fall heights and flow rates. Hydraulic stress and pressure exert a major influence on head cut mechanics, as well as on conventional hydraulic structure performance. Stress and pressure were measured in time and space on the boundary of an over fall using hot-film anemone try and pressure transducers. (7 references)

112. K.M. Robinson, 1992. "Predicting Stress and Pressure at an Overfall." Transactions of the American Society of Agricultural Engineers. 35(2):561-569.

Hydraulic shear stress and pressure forces have a major influence on the development and movement of a gully headcut or overfall. These forces were measure in a straight drop overfall model, and generalized prediction equations were developed using the dominant hydraulic and geometric parameters. The relatively simple model discussed herein provide a means of predicting the magnitude and variance of stress and pressure on the boundary of an overfall. Reasonable estimates of boundary shear stress allow estimation of headcut movement.

113. K.M. Robinson, 1992. "Gully Headcut Submergence." ASAE paper 922004, 11 pp. (Mimeo handout)

This paper presents results of a model study on gully hydraulics. Three dimensional hydraulic changes are examined as a gully moves through a horizontal reach while subjected to a constant flow. The conditions necessary to submerge a gully headcut are the primary focus of the investigation.

114. K.M. Robinson, G.J. Hanson, 1994. "A Deterministic Headcut Advance Model." Transactions of the American Society of Agricultural Engineers. Vol. 37(6):1437-1443.

Gully erosion is a dominant form of damage to earth spillways. Prediction of the rate of gully headcut movement is desired to assess the risk of a spillway breach. This article presents a headcut advance model composed of a boundary stress prediction model and a mass failure model. Several simplifying assumptions were necessary to predict headcut advance. Model performance was compared with two cases of field damage.

115. K.M. Robinson, G.J. Hanson, 1995. "Large Scale Headcut Erosion Testing." Transactions of the American Society of Agricultural Engineers. 38(2), 429434.

The development and movement of gully headcuts can cause major damage in earth emergency spillways. A 1.8-m-wide and 29-m-long flume with 2.4-m-high side walls was constructed to perform research on headcut advance. Headcut advance tests were conducted holding discharge, overfall height, and backwater level constant while varying soil properties. Two soil types were examined, and the soil properties were altered by compacting the material in the flume at varying moisture and density conditions. The observed headcut advance rates varied by a factor of more than 100 depending on the placement conditions.

116. K.M. Robinson, C.E. Rice, K.C. Kadavy, 1995. "Stability of Rock Chutes." Proceedings of the First Interagency Conference. Water Resources Engineering. ASCE. (2): 1476-1480.

The stability of rock chutes was examined in three separate flumes. Predominantly angular stone with a D_{50} ranging from 15 to 155 mm was tested on slopes of 10, 12.5, 16.6, 22.2, and 40%. Chute stability increased as the stone size increased and as the bed slope decreased. An empirical relationship was developed to predict the highest stable unit discharge as a function of material D_{50} and the bed slope.

117. A. Robles, 1983. "Design and Performance of Stabilizers and Drop Structures." (unpublished document)

The US Army Engineer District, Los Angeles, has made extensive use of channel stabilizers and drop structures in the design of major flood control channels in southern California. These include the Los Angeles River and the San Gabriel River Channels. This paper discusses the difference in design concept between the two and presents results of hydraulic model studies and construction techniques, as well as performance of existing structures.

S

118. Saunders, Peter A, and Grace, John L., Jr. 1981. "Channel Control Structures for Souris River, Minot, North Dakota; Hydraulic Model Investigation." Technical Report HL-81-3, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Model tests of channel control structures constructed of concrete and gabions were conducted at an undistorted scale ratio of 1:12 to determine the discharge characteristics of the structures and size and extent of riprap required to prevent scour downstream of the structures, effects of ice flowing over the structures, and stability of the gabion structures. The gabion structure was located in a typical section of trapezoidal channel and a stable gabion configuration was developed by extending the gabions farther up the side slopes and farther downstream of the structure than was indicated in the original design. The concrete structure was placed in an expanded section of trapezoidal channel with riprap protection on the side slopes and on the channel bottom upstream and downstream of the structure. Model results indicated that the original size and extent of protection could be reduced without endangering the structure. Free and submerged flow discharge characteristics were determined for both types of channel control structures tested, and stability criteria were developed for the gabion structures.

119. K. Senoo, T. Mizuyama, 1983. "Evaluation of a Sabo Dam as a Countermeasure Against Debris Flow." Proceedings of the Twentieth Congress of the International Association for Hydraulic Research. Moscow, USSR. September 5-9, 1983. Seminar 2, Vol. VII, Paper S.2.1. pp. 280-285.

This paper describes the characteristics of two types of sabo (erosion control or check) dams to reduce sediment discharge from debris flows. Flume studies are used to examine sediment discharge behavior in silt and wall type dams. (3 references)

120. F.D. Shields, Jr., M.R. Palermo, 1982. "Assessment of Environmental Considerations in the Design and Construction of Waterway Projects." Technical Report E-82-8, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Waterway projects covered in this report include channel modifications for flood control, navigation, dikes, stream bank protection, and levees. Flood control channel modifications include clearing and snagging;

channel enlargement, alignment, and relocation; and channel stabilization using grade control structures or stream bank protection. Adverse environmental impacts of flood control channel modification include loss of valuable habitats and habitat diversity, channel instability, reduction of aesthetic value, water quality degradation, and undesirable hydrologic changes. Immediate and eventual losses of backwater habitat are a major impact of navigation channel modification projects. The major environmental impact associated with dikes is the reduction in water-surface area and loss of habitat diversity due to sediment accretion in the dike field. Major adverse effects of stream bank protection include loss of riparian vegetation and reduction in the rate of channel migration. The major environmental impact of levees is related to their purpose: the creation of drier conditions on the land side of the levee is frequently associated with land use changes. Recent efforts to incorporate environmental considerations in levee projects include management of vegetation on and around levees for wildlife and aesthetics and recreational features. (159 references)

121. F.D. Shields, Jr., J.J. Hoover, N.R. Nunnally, K.J. Killgore, T.E. Schaefer, T.N. Waller, 1990. "Hydraulic and Environmental Effects of Channel Stabilization, Twentymile Creek, Mississippi." Technical Report EL-90-14, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Twentymile Creek, located in northeast Mississippi, was straightened and enlarged about 1910, 1936-37, and 1966. Extreme channel instability followed the 1966 modifications, and corrective measures (placement of bank protection and construction of three grade control structures) were taken between 1982 and 1988. Hydraulic and environmental studies were performed to determine effects of the corrective measures.

122. F.D. Shields, Jr., J.J. Hoover, 1991. "Effects of Channel Restabilization on Habitat Diversity, Twentymile Creek, Mississippi." Regulated Rivers: Research & Management, Vol. 6, 163-181.

Twentymile Creek, a sand-bed stream draining a 450 km^2 catchment in northeast Mississippi, was channelized prior to 1910, in 1938, and in 1966. Straightening and enlargement in 1966 was followed by channel instability-rapid bed degradation (2-4 m) and cross-section enlargement by 1-4 to 2-7 times. Grade control structures (GCS) (Weirs with stone-protected stilling basins)and various types of stream bank protection were constructed along the channel in the early 1 980s to restore stability. Other investigators have suggested that habitat recovery in incised, channelized streams is facilitated by construction of GCS because they create stable scour holes and promote natural formation of a low-flow channel flanked by vegetated berms. Effects or restabilization of Twentymile Creek on aquatic habitats were assessed in four ways. The fraction of the bank line covered by woody vegetation was mapped from aerial photographs taken in 1981 and 1985; physical habitat (depth, velocity, substrate, and cover) and fishes were sampled at base flow; and the existence and size of a low-flow channel was ascertained from cross-section surveys taken in 1980 and 1989. Woody vegetation, physical aquatic habitat, and fishes were also sampled from Mubby-Chiwapa Creek, a similar-sized unstable channel with no GCS. Physical habitat variables and fishes were sampled concurrently at five stations on Twentymile Creek, and four stations on Mubby-Chiwapa. Four of the five Twentymile stations were either above or below a GCS. Bank-lined woody vegetation cover increased 8 percent between 1981 and 1985 along Twentymile Creek but was stable along Mubby-Chiwapa. Reaches above and below GCS were deeper with slower current velocities than elsewhere. Mean Shannon diversity indices based n physical data were similar for both streams, but were 58 percent higher for stations immediately above and below GCS than for other stations. Since construction of the GCS and bank protection measures, longitudinal berms have formed within the enlarged Twentymile Creek channel, creating a low-flow channel. Low-flow channel capacity was equivalent to a mean daily discharge equaled or exceeded 30 percent of the time, and was considerably lower than the effective discharge. Differences in aquatic habitat diversity among the stations sampled were primarily due to the scour holes below the GCS and the low-flow channel. Thirty-nine fish species were collected from Twentymile Creek, but only 22 from Mubby-Chiwapa. Fourteen species were collected exclusively at GCS. Principal component analyses of the abundance of the eight numerically dominant fish species indicated similar faunas at most stations, but Twentymile Creek. GCS stations were faunistically distinct. Abundance of several of the numerically dominant species was positively influenced by greater depths and lower velocities found near Twentymile GCS. The mean fish diversity index for

Twentymile Creek was 29 percent higher than for Mubby-Chiwapa, and fish diversity was positively correlated with substrate diversity and mean depth.

123. F.D. Shields, Jr., S.S. Knight, C.M. Cooper, 1996. "A Tale of Two Streams: Restoration Strategies Compared." Proceedings 1996 Fed Interagency Sedimentation Conference. Las Vegas, Nevada.

Many stream ecosystems are severely limited by damaged physical habitat. Channelization and associated accelerated erosion is a primary cause of damages in agricultural watersheds. Cost-effective strategies are needed to address erosion problems and restore stream corridor habitats. Detailed studies of restoration outcomes are rare. herein we present a case study of two small streams (watershed size = 12 and 14 cm^2) damaged by channel straightening and incision. One stream was stabilized using a low drop grade control structure and dormant willow post planting, while the other was treated with a stone weir, stone toe bank protection, and willow sprout planting. Effects of restoration were monitored by collecting physical and biological data for one to two years before restoration and two to three years afterward. Following construction, channel plan forms were stable, but up to 1 m of deposition and erosion occurred along the thalweg profile. Willow planting was not successful, so canopy, bank vegetation and woody debris density were unchanged. Pool habitat areas increased from less than 5% to more than 30%. Fish species richness was unchanged, but species composition shifted away from cyprinids that occur in shallow, sandy runs toward pool-dwelling types (catostomids and centrarchids). Response to restoration was more modest than for two nearby restoration projects. Potential causes include less ambitious restoration design, greater initial degradation, and isolation from less-degraded sites, which could serve as sources of colonists.

124. C.C. Shih, D.F. Parsons, 1967. "Some Hydraulic Characteristics of Trapezoidal Drop Structures." Proceedings of the Twelfth Congress of the International Association for Hydraulic Research. Fort Collins, CO. September 11-14, 1967. Colorado State University. Vol. 3. pp. 249-259.

This study is concerned with some of the hydraulic characteristics of flow over drop structures connecting two trapezoidal channels at different elevations. The drop structures are formed by an abrupt drop in the horizontal channel bottom, and are equipped with or without a weir. The relationships among the quantities pertinent to the hydraulic characteristics are investigated through dimensional and experimental analyses. The dimensional analysis, which was based on theoretical consideration of the flow problem, resulted in a set of dimensionless parameters, namely the relative depth, the drop number, and the geometrical parameters of the drop structure. Experimental results are presented in graphical form through dimensionless parameters for various trapezoidal drop structures. Some typical flow phenomena are shown in photographs. (9 references)

125. C.D. Smith, 1985. Hydraulic Structures, University of Saskatchewan Printing Services, Saskatoon, Canada.

Contents: Chapter l Storage Dams (19 references); Chapter II: Spillway (26 references); Chapter III: Outlet Works (15 references); Chapter IV: Gates and Valves (11 references); Chapter V: Division Works (16 references); Chapter VI: Drop Structures (17 references): Chapter VII: Stone Structures (19 references); Chapter VIII: Conveyance and Control Structures (12 references); Chapter IX: Flow Measurement (16 references); Chapter X: Culvert Hydraulics (32 references); Appendix 1: SI Units; Appendix 2: Answers to Problems.

126. C.D. Smith, D.G. Murray, 1975. "Cobble Lined Drop Structures." Canadian Journal of Civil Engineering. Vol. 2, No. 4. pp. 437-446.

This paper describes the concept and the experimental study made to determine the design criteria for a drop structure comprised entirely of loose dumped rock. The three principal components of the design were (1) the weir at the top of the slope, (2) the sloping protection downstream from the weir, and (3) the horizontal apron at the bottom. The rock weir was intended to prevent excessive draw down and control upstream velocities. The sloping protection was a hydraulically steep reach, on which supercritical velocity occurred. The apron was intended to accommodate the transition back to subcritical flow at the end of the drop structure. The initial investigation involved study of the structure in two-dimensional flow. It was determined that the critical area for stability was in the terminal velocity region of the steeply sloping portion of the structure. The stone size and layer thickness required for channel stabilization were found to be a function of the channel slope and flow depth at terminal velocity. Eight combinations of variables were tested and included four different slopes, three different stone sizes, and two different layer thicknesses.

The failure process was unique in that there was an initial failure and an ultimate failure. Initial failure occurred when some stone at the lower end of the slope was displaced, exposing the subgrade. The exposed area was rapidly filled in by downstream migration of stones from further up the slope. The migration process continued until it was arrested at the top of the slope by the rock weir, and the structure became stable once again. A further large increase in discharge was required to precipitate a second or ultimate failure.

Design criteria were formulated, including a recommended factor of safety based on initial failure, and the design was verified on a three-dimensional model. (9 references)

127. C.D. Smith, D.K. Strang, 1967. "Scour in Stone Bed." Proceedings of the Twelfth Congress of the International Association for Hydraulic Research. Fort Collins, CO. September 11-14, 1967. Colorado State University. Vol. 3. pp. 65-73.

When stone of suitable quality and size is available near the proposed site of a hydraulic structure, the possibility of using such stone for part of the structure exists. For example, a stone-lined stilling pool may be substituted for a more costly reinforced concrete stilling basin. Scour due to nappe impingement in a stone bed downstream from a vertical drop structure was studied. The stone size and areal extent necessary for a dependable design was determined. The data are presented in dimensionless charts.

128. J.P. Soni, R.J. Garde, K.G. Ranga Raju, 1980. "Aggradation in Streams due to Overloading." Journal of the Hydraulics Division, ASCE, Vol. 106, No. HYl, January, 1980. pp. 117-132

The problem of aggradation due to increase in the rate of sediment supply in excess of what the stream can carry has been investigated. The supply of sediment is assumed to be continuous and at a constant rate. A relationship for the depth of aggradation at any time and at any distance from the section of sediment addition has been developed. Since the mathematical model used was based on many simplifying assumptions, it needed verification against a known set of data. Experiments were, therefore, performed in the laboratory and these have enabled suitable modification of the analytical results.

129. C.N. Strauser, C.W. Denzel, 1984. "Geomorphic Changes and Grade Stabilization of the Kaskaskia River." River Meandering. Proceedings of the ASCE Conference Rivers '83. New Orleans, LA. October 24-26, 1983. Charles M. Elliott, ed.. pp. 410-417.

Design studies for the Kaskaskia River grade control structure at the head of the navigation channel on the river near the US Highway 460 bridge are described. The structure permits re-dredging of the upper navigation canal without upstream bank erosion and aggradation of the downstream channel bottom. Studies performed include a HEC-2 backwater model study (to obtain water-surface profiles) and a physical model study to determine discharge characteristics, water-surface elevations, etc. The proposed riprap appears adequate. (1 reference)

130. W.A. Stufft, 1965. "Erosion Control for Gering Valley." Proceedings of the ASCE Hydraulic Engineering Conference. Tucson, AZ.

A comprehensive plan of improvement was developed by the Soil Conservation Service and the Corps of Engineers to control erosion in Gering Valley, Nebraska. General features of the plan are described with special emphasis on the development of grade stabilization structures by the Corps. Engineering judgement and experience were heavily relied on because complete technical criteria were not available.

T

131. W.C. Taggart, J.M. Pflaum, E.A. Stiles, B. DeGroot, 1987. "Evaluation of and Recommendations for Drop Structures in the Denver Metropolitan Area." Hydraulic Engineering Proceedings of the ASCE National Conference on Hydraulic Engineering. Williamsburg, VA. August 3-7, 1987. Robert M. Ragan, ed.. pp. 19-24.

Numerous partial failures of drop structures constructed in the Denver metropolitan area as components of channel improvements constructed for flood control and in conjunction with urbanization, many resulting from flows less than design flows, led the Denver Urban Drainage and Flood Control District to consider a complete evaluation of drop structures. This paper discusses the first phase of the work, which culminated in a comprehensive report that was presented to the District, local government representatives, and engineering consultants at a seminar conducted in Denver on December 12, 1986.

132. C.H. Tate, 1987. "Muddy Creek Grade Control Structures, Muddy Creek, Mississippi and Tennessee." Proceedings of the Seventeenth Mississippi Water Resources Conferences. Jackson, MS. March 25-27, 1987. Mississippi Water Resources Institute, Mississippi State University, Starkville, MS. pp. 63-67. (Also as Technical Report HL-88-11, USACE WES, Vicksburg, MS.)

Between September 1976 and September 1983, the Soil Conservation Service modified the Muddy Creek system by constructing a trapezoidal channel with 12 riprap grade control structures spaced along the main channel. Flow separation with resultant flow concentration in the exit transitions was determined to be the reason for scour downstream of the grade control structures. Tests were conducted to determine what modifications were required to these existing grade control structures that have 1 on 4 and 1 on 8 exit flares to reduce or eliminate significant scour problems previously observed at these structures. Since the exit flares were fixed, different types of modifications involving baffle piers or a hump placed in the exit transition were tested in an attempt to produce a uniform distribution of flow at the end of the grade control structure. A baffle arrangement with the height of the baffle piers being 75 percent of the design depth was the most effective design in producing a uniform distribution of flow in the exit channel without any significant backwater effect in the grade control structure. For this type of grade control structure without the use of baffles, flow separation occurred at the upstream end of the exit transition if the exit flare was greater than a 1 on 12 ratio. Minor irregularities (differential settlement or vegetation) on the side slopes of a 1 on 12 flare caused separation and flow concentration, indicating that this was approximately the critical flare ratio below which incipient flow separation occurs. Additional tests indicated that the 1 on 16 exit flare was the maximum that provided satisfactory flow conditions without being sensitive to minor irregularities on the side slopes, and therefore was the recommended design. (2 references)

133. Y-K. Tung, L.W. Hays, 1982. "Optimal Design of Stilling Basins for Overflow Spillways." Journal of the Hydraulics Division, ASCE, Vol. 108, No. HYlO, October, 1982. pp. 1163-1178.

A model has been developed for use in determining the optimal dimensions of stilling basins for overflow dam spillways and their appurtenances that minimize cost and satisfy hydraulic performance. The decision variables which are the width and elevation of the stilling basins are determined such that the hydraulic performance is satisfied for a wide range of possible spillway discharges. The model is based upon an optimization procedure with two phases. The first phase is used to optimize the hydraulic performance of a stilling basin and the second phase 15 to use the first phase results to minimize the cost satisfying required basin performance. Both phases are based upon a gradient search technique. Application of the optimization model was made using data for the spillway and stilling basin designs for three dams in Texas.

134. H.O. Turner, Jr., 1988. "Sweetwater River Channel Improvement Project, San Diego County, California; Hydraulic Model Investigation." Technical Report HL-88-3, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

In the area of proposed improvement, the Sweetwater River is a poorly defined channel varying from 1,200 to 2,000 ft wide in a relatively broad flood plain. An entrenched trapezoidal channel with a base width of 320 ft has been excavated ending just upstream of a freeway bridge. This channel has a radius of 1,000 ft and turns approximately 80 degrees in relation to the proposed channel alignment through the freeway bridge. A drop structure is to be located in the radius of the curve at the beginning of the project. A study of the proposed project was conducted using a fixed bed constructed at a scale of 1:40 to study the effect of downstream waves and disturbances caused by the curvilinear flow conditions. The main objectives of the study were to obtain quantitative information on flow patterns, flow distribution, waves, and disturbances throughout the curved reach of channel, as well as to determine the effects of sediment buildup on water-surface elevations. The model study revealed that certain refinements are needed to the Sweetwater River project to eliminate potential problems.

135. H.O. Turner, Jr., M.E. Mulvihill, 1987. "General Design for Modifications to Existing Lower Santa Ana Drop Structures." Hydraulic Engineering, Proceedings of the ASCE National Conference on Hydraulic Engineering. Williamsburg, VA. August 3-7, 1987. Robert M. Ragan, ed.. pp. 1118-1123.

A model study conducted at the US Army Engineer Waterways Experiment Station investigated the possibility of utilizing the existing drop structures of the Santa Ana River to provide flood protection for the lower basin. The results of this model study show that the existing drop structures will not adequately convey the increased discharges expected without severe scour to the stream bed or structural failure. Different modifications tested included a parabolic crest, baffle blocks, and a sloping end sill. (6 references)

136. D. Twiss, 1985. "California Rock Drop." Proceedings of the West States Engineering Workshop. Portland, OR. February 4-8, 1985. US Department of Agriculture, Soil Conservation Service. Davis, CA (unpublished).

California rock drops have been used as energy dissipaters reducing the energy gradient to provide a stable channel.

U

137. US Army Corps of Engineers. 1970. Hydraulic Design of Flood Control Channels." EM 1110-2-1601, US Government Printing Office, Washington, DC.

Drop structures are designed to check channel erosion by controlling the effective gradient, and to provide for abrupt changes in channel gradient by means of a vertical drop. They also provide a satisfactory means for discharging accumulated surface runoff over fills with heights not exceeding about 5 ft and over embankments higher than 5 ft provided the end sill of the drop structure extends beyond the toe of the embankment. The hydraulic design of these structures may be divided into two general phases, design of the notch or weir and design of the overpour basin. Drop structures must be so placed as to cause the channel to become stable. The structure must be designed to preclude flanking. (94 references)

138. US Army Engineer District, Buffalo. 1975. "Design Analysis: Energy Dissipater Facilities and Riprap Repair, Coy Glen and Cayuga Inlet, Ithaca, New York." Buffalo, NY.

The design analysis for this project provides: (1) two hydraulic drop structures and attached wing wall on Coy Glen; (2) soils and foundation analysis for the above structures and cantilever sheet pile wing wall alternates for the two drop structures; (3) riprap repair for the section in Cayuga Channel between the Lehigh Valley Railroad bridge and the drop structure at Station 160+00; and (4) dynamic water loads on the drop structure and hydraulic design for Coy Glenn by the Buffalo District. The design considered two types of wing walls. The factor of safety in bearing for the concrete wing walls is not considered adequate, and the more conservative steel sheet pile wing walls are recommended.

139. US Army Engineer Division, North Pacific. 1983. "Libby Reregulating Dam, Kootenai River, Montana." Technical Report 160-1, USAE Division Hydraulic Laboratory, Bonneville, OR.

The proposed Libby Reregulating Dam would be located on the Kootenai River 10 miles downstream from Libby dam. One of the principal features of the project is a combination of the spillway and powerhouse in a single structure. Design of the spillway/powerhouse was verified using three models-a 1:5-scale spillway model, a 1:35.33-scale spillway/ powerhouse sectional model, and a 1:80-scale comprehensive model. Tests in both the sectional and comprehensive models were accomplished using both the sectional and comprehensive models were accomplished using both fixed- and moveable-bed boundaries. During early stages of project design, a baffled-chute spillway was developed which provided capability for reduction in nitrogen supersaturation. Evaluation of the design accomplished in a 1:25.11-scale model is included as an appendix to this report. The spillway exhibited adequate energy dissipation for discharges up to 903 $ft^3/s/ft$ and potential for reduction in gas supersaturation for discharges up to 181 $ft^3/s/ft$. The concept was not pursued in final design due to economics combined with operating considerations at the upstream Libby Dam.

140. US Department of Agriculture. 1975. "Bryant Swamp Watershed, Bladen County, North Carolina." Soil Conservation Service, Raleigh, NC.

Bryant Swamp Watershed, located in the southwestern part of Bladen County, North Carolina, has an area of 16,200 acres. Project measures include land treatment, 22.9 miles of stream channel modification, and six grade control structures. Environment effects include the following: (1) reduced flooding on cropland, forested alluvial flood plain, and in the town of Bladenboro; (2) improved solid profile drainage; (3) reduce erosion; (4) create 40 acres of wildlife food and cover; (5) provide better mosquito control; (6) create 12 jobs during construction and g jobs for the life of the project; (7) reduce value of 50 acres of wildlife wetland habitat; (8) damage one mile of fishing stream; (9) increase sediment during construction; (10) clear 95 acres of woodland; and (11) damage 90 acres of woodland. (3 references)

141. US Department of Agriculture. 1973. "Burnt Creek RC&D Measure Plan for Flood Prevention, Lewis and Clark 1805 Resource Conservation and Development Project, Burleigh County, North Dakota." Soil Conservation Service, Bismarck, ND.

Works of improvement consist of a floodwater diversion, dikes, grade control structure, a structure to divert low flows to Burnt Creek, and an inverted siphon to carry irrigation water across the diversion. The project will reduce flooding on about 2,500 acres of agricultural land and a sparsely settled rural residential area; destroy 2.5 acres of woody and herbaceous cover which will be mitigated by dedicating 5 acres of similar habitat for the life of the project; provide for maintenance of the essential integrity of the existing Burnt Creek channel; provide for use of the flood diversion berm and dikes by wildlife; and provide maintenance of the floodway below the diversion (an old Missouri River channel and appurtenant dikes) in such manner as to be beneficial to project purposes and enhancement and preservation of wildlife cover.

142. US Department of Agriculture. 1974. "Chicod Creek Watershed, Pitt and Beaufort Counties, North Carolina, Environmental Statement (Revised)." Report No. USDA-SCS-WS-ES-(ADM)-72-27 (in 4 volumes), Solid Conservation Service, Raleigh, NC.

The project is concerned with Chicod Creek Watershed which is located in Pitt and Beaufort Counties, North Carolina. Project measures include land treatment, 66 miles of stream channel modification, 2 wildlife wetland preservation areas, l warm water impoundment, 11 rock structures, 30 water-control structures, and 10 sediment traps. A summary of environmental impact and adverse environmental effects is given. (63 references)

143. US Department of Agriculture. 1971. "Criteria for the Hydraulic Design of Impact Basins Associated with Full flow in Pipe Conduits." Technical Release No. 49, Soil Conservation Service, Washington, DC.

This technical release presents the recommendations on impact basins taken from the Bureau of Reclamation publication Hyd-572, "Progress Report No. XIII, Research Study on Stilling Basins, Energy Dissipaters, and Associated Appurtenances; Section 14, Modification of Section 6 (Stilling Basin for Pipe or Open Channel Outlets-Basin VI)." dated June 1969, by G. L. Beichley. This release presents these recommendations as criteria for impact basins associated with the flail pipe flow and pipe diameters from 1.5 to 5.5 ft, inclusive. The user may obtain the proportioning of the impact basin and the sizing of the required riprap form the included drawings.

144. US Department of Agriculture. 1977. "Design of Open Channels." Technical Release No. 25, Soil Conservation Service, Washington, DC.

This release covers procedures for design of open channels and related measures such as floodways. Criteria and standards applicable for each situation should be used in conjunction with these procedures. Designers of open channels should find this release useful in considering the numerous technical aspects that are important to sound channel modifications. Close coordination of many technical fields is important if channels with minimum environmental impacts are to be developed.

145. US Department of Agriculture. 1983. "Design and Analysis of Rock Chutes." Design Note No. 22, Soil Conservation Service, Washington, DC.

This design note presents a simple procedure for the design of riprap chutes. The note has been developed from established data and theory regarding hydraulic resistance and rock stability to make available a technically sound method for the design of riprap chutes. The theoretical basis of the method used a friction formula for flows along boundaries with discrete and movable roughness elements and a stability relationship prescribing the condition of incipient movement of these elements.

146. US Department of Agriculture. "Drop Spillways." Engineering Handbook, Section 11, Soil Conservation Service, Washington, DC.

This handbook presents in brief and usable form information on the application of engineering principles to the problems of soil and water conservation. The scope is necessarily limited to phases of engineering which pertain directly to the program of the Soil Conservation Service. Therefore, emphasis is given to problems involving the use, conservation, and disposal of water, and the design and use of structures most commonly used for water control. Typical problems encountered in soil and water conservation work are described, basic considerations are set forth, and all of the step-by-step procedures are outlined to enable the engineer to obtain a complete understanding of a recommended solution.

147. US Department of Agriculture. 1976. "Hydraulic Design of Riprap Gradien Technical Release No. 59 with Supplements 1(1976) and 2 (1978) and Amendment 1 (1986), Soil Conservation Service, Washington, DC.

This technical release presents the criteria and procedures for the design and proportioning of riprap gradient control structures (RGCS). It is a structure consisting of a prismatic channel with a converging inlet transition at the upstream end and a diverging outlet transition at the downstream end of the prismatic channel. Its essential feature is that the specific energy of the flow at design discharge is constant throughout the structure and is equal to the specific energy of the flow in the channel immediately upstream and downstream of the structure. Thus, the dissipation of hydraulic energy in the structure is at the same rate as energy gain due to the gradient. The structure, which is made steeper and narrower than the adjoining channel upstream and downstream, maximizes energy dissipation. A computer program, written in FORTRAN, determines dimensions and parameters associated with the design of an RGCS.

148. US Department of Agriculture. 1985. "Hydraulics of Two-Stage Straight Drop Spillways." Report No. PB85-174688, Soil Conservation Service, Washington, DC.

The report identifies the five flow regimes likely to occur during the passage of outflow hydrographs over two-stage straight drop spillways and presents a method for developing rating curves for such structures under any flow conditions. The advantages of the method are fully appreciated only in applications where at least one of the two stages is susceptible to submergence. The components of the compound section delineating the crest of the spillway may be rectangular or trapezoidal weirs. Satisfactory performance of the structure, in regard to energy dissipation, dictates symmetrical configuration of the compound section about the axis of the lower stage. The method can be used in the development of rating curves for existing structures not meeting the aforesaid performance requirement. (11 references)

149. US Department of Agriculture. "Hydraulics." Engineering Handbook, Section 5, Soil Conservation Service, Washington, DC.

This handbook presents in brief and usable form information on the application of engineering principles to the problems of soil and water conservation. Emphasis is given to problems involving the use, conservation, and disposal of water, and the design and use of structures most commonly used for water control. Typical problems encountered in soil and water conservation work are described, basic considerations are set forth, and all of the step-by-step procedures are outlined to enable the engineer to obtain a complete understanding of a recommended solution.

150. US Department of Agriculture-SCS-Wisconsin, Section IV, Technical Guide. Grade stabilization structure. 1994.

The article present generalized design criteria for structures to control the grade and head cuffing in natural or artificial channels.

V

151. V.A. Vanoni, R.E. Pollack, 1959. "Experimental Design of Low Rectangular Drops for Alluvial Flood Channels." Report No. E-82, California Institute of Technology, Pasadena, California.

This report describes research work done under contract with the Berkeley Office of the US Department of Agriculture, Soil Conservation Service, on the use of rectangular drops as grade stabilizers in alluvial channels. Runs were made in two flumes-one with a fixed bed and clear water, the other with an erodible bed and sediment-carrying water. Two cases were investigated in each flume: drops on mild slopes and drops on steep slopes. The dimensions of the drops were varied to establish the combinations which give the best performance.

The principal laboratory results are as follows:

1. Drops behave differently with sediment-laden flow than they do with clear flow. The sediment changes the flow pattern and enables the formation of sand waves in the channel.

2. Larger basins are required for sediment-laden flow than for clear flow.

3. The size of the scour hole downstream from a drop increases with the Froude number.

4. The dimensions of the basins for satisfactory performance increase with the Froude number.

Design curves and sample calculations are presented. The complete tabulated data are presented in the appendix. (9 references)

W

152. Water Engineering & Technology, Inc., 1988. "Performance Evaluation of Channels Stabilized with ARS-Type Low-Drop Structures." Final Report Submitted to the U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, July 1988.

This report contains the results of an evaluation of the success of channel rehabilitation by the use of AILS-type low-drop grade control structures. The basic design for these structures was developed in a series of physical model tests which began in 1974 at the AILS Sedimentation Lab in Oxford, MS (Little and Murphy 1982, item 55). These structures have been used extensively in northern Mississippi by the US Army Engineer District, Vicksburg, and the Soil Conservation Services, US Department of Agriculture. Although the model tests provided extensive data about the structure design, evaluation of field performance and response of the channel system to the structures was beyond the scope of the developmental studies. The objectives of this investigation were to (1) compare the previous model study results with field operation, (2) determine the effect of the structures on channel aggradation and degradation, (3) determine the effect of the structures on channel bank stability, and (4) ascertain the validity of using a design discharge less than the maximum channel flood capacity. (22 references)

153. J. Whittaker, A. Schleiss, 1984. "Scour Related to Energy Dissipaters for High Head Structures." Mitteilungen der Versuchsanstalt fur Wasserbau. Hydrologie und Glaziologie, Nr. 73, an der Eidgenossischen Technischen Hochschule Zurich, 1984.

Background theory is presented on predicting jet trajectories and behavior in air, as well as on the characteristics of a plunging jet in water. The role of model tests in predicting scour is discussed, and some difficulties relating to grain size effects noted. Predicting scour caused by horizontal jets issuing from energy dissipation basins and by plunging jets from free overfall, pressure outlet or ski-jump spillways is then covered in some depth.

154. J. Whittaker, M. Jaggi, 1986. "Blockschwellen." Mitteilungen der Versuchsanstalt fur Wasserbau Hydrologie und Glaziologie, Nr. 91, an der Eidgenossischen Technischen Hochschule Zurich, 1986 (in German).

The design of block ramps, which are used to stabilize river beds against erosion, cannot be based on established rules. Laboratory tests showed, that direct erosion, embedding of blocks or sliding on the underlying material, and the scour at the end of the ramp can lead to destruction individually or in combination. These tests also allowed design criteria for each case to be defined. Direct erosion of blocks out of a ramp is a particular case of the problem of incipient motion of movable river bed material. Equation (35), which is in agreement with the general theory concerning this problem, allows the maximum acceptable specific discharge to be calculated. Equation (46) must also be considered to make ramps safe against embedding and sliding. These equations are valid for uniform flow. If the flow on the ramp does not become uniform, then smaller stresses occur along the ramp. A detailed hydraulic analysis may result in considerable reductions of these maximum stresses compared to uniform flow, specially for high specific discharge and small fall heights. Scour dimensions are also strongly dependent on the hydraulic conditions. It is absolutely necessary that a backwater effect always extends from downstream over the toe of the ramp, to prevent the last blocks from sliding into the scour hole. A modified form of the Tschop/Bisaz formula (equation 58) allows an approximation for the scour depth to be found. However, three-dimensional flow effects may have to be considered as well.

155. R.F. Wong, A. Robles, Jr., 1971. "Flood-Control Facilities for Unique Flood Problems." Journal of the Waterways and Harbors Division, ASCE, Vol. 97, NO. WW1. pp. 185-203.

The unusual climatic, hydrologic, topographic, and physiographic conditions in southern California are discussed. The unusual conditions include extreme concentration of seasonal rainfall and runoff, short-duration and high-peak storms, steep topographic gradients, and combination of physiographic and cultural characteristics. Facilities include debris basins, concrete-paved channels, earth channels and levees with and without grade control structures, and continuous single levees.

156. D.A. Woolhiser, A.T. Lenz, 1965. "Channel Gradients Above Gully-Control Structures." Journal of the Hydraulic Division, ASCE, Vol. 91, No. HY3. pp. 165-187.

Regression techniques are used to determine some significant factors affecting the slope of the sediment deposition profile above gully control structures and debris dams. Analyses of field data on gully control structures in Wisconsin and debris dams in California show that the original channel slope, the width of the channel at the structure, and the height of the spillway crest above the original thalweg are statistically significant variables. (20 references)

157. J.T. Wulliman, K.D. Hansen, 1990. "Design and Construction of Soil-Cement Drop Structures." Proceedings of the ASCE Hydraulic Engineering Conference. July 30, 1990. San Diego, CA.

Soil-cement has been used as an effective and economical slope protection material for embankment dams and stream banks. This paper describes the use of soil-cement in the construction of urban storm water drop structures, a relatively new application. In locations where an adequate supply of in-place sandy soil is available, soil-cement offers advantages in terms of economy and aesthetics over other drop structure construction materials. This paper describes the design and construction of seven soil-cement drop structures recently built in the Denver, Colorado, area.

REFERENCES

<What needs to be said. I have so many I don't know what to do with them all. The hardest part is keeping the list small and to stay on subject. This list will replicate the annotated bibliography plus all other references from the text. I want to include a cited and "other" references section here.>

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2. J.H. Ables Jr., G.A. Pickering, 1975. "Flood Control Project on Lytle and Warm Creeks and Santa Ana River, California; Hydraulic Model Investigation." Technical Report H-75-7, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

3. J.H. Ables Jr., 1976. "Divide Cut Drainage Structures, Tennessee-Tombigbee Waterway, Mississippi and Alabama; Hydraulic Model Investigation." Technical Report H-76-18, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

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